SAHC

ADVANCED MASTERS IN STRUCTURAL ANALYSIS
OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master's Thesis

Nilma del Coral Muñiz Acevedo

Vulnerability Analysis of Timber Frame Walls Under Seismic Loading

UNIVERSITAT POLITÈCNICA DE CATALUNYA

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Education and Culture

Erasmus Mundus

Vulnerability Analysis of Timber Frame Walls Under Seismic Loading

Nilma del Coral Muñiz Acevedo

Spain | 2018

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ABSTRACT

In many regions of the world, the process of design and construction varies depending on local needs and conditions, availability of construction materials, and traditional construction techniques. Timber frames are an example of vernacular architecture with historical and cultural significance in many regions, like: Europe, Latin-America, North America and Oceania. The extensive diffusion of this type of construction has caused the development of several structural typologies and solutions.

This master thesis studied three vernacular timber frame typologies in order to determine their resistance and behavior toward seismic actions. To obtain this aim, numerical analyses were performed by including variations in material, geometry, bracings and carpentry joints types. Nonlinear static (pushover) analysis following the lumped plasticity modelling approach were carried out to determine the structural capacity of the frames. The complex behavior of the carpentry connections was studied to carry out a modelling calibration based on previous experimental works. The results of these analyses contributed to understand the global behavior and the structural response of three timber frame typologies.

A numerical model was developed for two traditional timber frame typologies from Valparaiso, Chile. The developed numerical model was based in the modelling calibration of two timber frames previously studied experimentally. The calibrated models were able to capture the stiffness of the timber frame walls, the deformation, the nonlinear behavior and the expected mechanisms that where described in previous experimental works. The analysis demonstrated the importance of the connections and their influence in the global behavior of the frame. It was understood, that the connections are the weakest location of the frame, where failure can occur. The work carried out in this study seek to search for a correct approximation to model timber frame structures in cases where there are no experimental studies that allow to obtain more accurate results from the numerical models. Since there are limited studies about the structural response of timber frame buildings under seismic loading, this master thesis seeks to contribute to their conservation and to motivate for their further research.

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RESUMEN

En muchas regiones del mundo, el proceso de diseño y construcción depende de las necesidades locales, la disponibilidad de materiales de construcción y de las técnicas constructivas tradicionales. Los entramados de madera son un ejemplo de arquitectura vernácula con un gran significado histórico y cultural en muchas regiones del mundo, como: Europa, América Latina, Norte América y Oceanía. La extensa difusión de este tipo de construcción a causado que se desarrollen diferentes tipologías estructurales.

Esta tesis de maestría estudiará tres tipologías vernáculas de entramados de madera con el fin de determinar su resistencia y comportamiento ante un evento sísmico. Para obtener este objetivo diferentes configuraciones estructurales se estudiarán a través de análisis numéricos; incluyendo las diferentes variaciones en material, geometría, arriostramientos y uniones tradicionales de carpintería. El enfoque del modelo numérico consistirá en un análisis estático no lineal con plasticidad concentrada que proveerá la capacidad estructural del entramado, lo cual contribuirá al entendimiento de su comportamiento global y respuesta estructural. El complejo comportamiento de las conexiones de carpintería fue estudiado para llevar a cabo un modelo calibrado basado en experimentos previos. Los resultados de estos análisis contribuyeron al entendimiento del comportamiento estructural de las tres tipologías estudiadas.

Un modelo numérico fue desarrollado para dos tipologías de entramados de madera de Valparaíso, Chile. Los modelos numéricos desarrollados fueron basados en la calibracion de dos entramados de madera previamente estudiados experimentalmente. Los modelos calibrados pudieron capturar la rigidez del entramado, la deformación, el comportamiento no lineal y el mecanismo esperado, según descrito en los experimentos previos. El análisis demostró la importancia de las conexiones y su influencia en el comportamiento global de la estructura, siendo esta la parte más frágil del entramado. El trabajo llevado a cabo en este estudio busca una aproximación correcta para modelar entramados de madera y que pueda ser aplicado a casos en donde no existe información experimental previa que permita obtener resultados mas exactos de los modelos numéricos. Ya que no hay una gran cantidad de estudios sobre la respuesta estructural de los entramados de madera ante carga sísmica, esta tesis de maestría busca contribuir a su conservación y a motivar a que se continúe desarrollando su estudio.

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CHAPTER 1 INTRODUCTION

1.1 Background and Motivation

In many regions of the world, the process of design and construction varies depending on local needs and conditions, availability of construction materials, and traditional construction techniques. Houses are one of the most common forms of vernacular architecture and is estimated that between 90% to 98% of the population of the world live in traditional non-engineered houses (Oliver, 2006).

Vernacular structures may result vulnerable when facing natural events, since they rarely comply with local codes. However, it has been demonstrated that some vernacular structural systems have a good behavior towards seismic events, since builders learned over centuries which construction techniques performed well and hence, modified newer structures (Quinn, 2017). Timber frame structures are one of these vernacular constructions. These types of constructions have gained interest for research after showing a good seismic resistant behavior in recent earthquake events like, the earthquake in Turkey in 1999, the one in India in 2005, and the one in Haiti in 2010.

Timber frame systems are one of the most common constructions in many regions of the world including: Europe, Latin-America, North America and Oceania. The extensive diffusion of this type of construction has caused the development of several structural typologies and solutions. Timber frame systems can vary according to the geometry of the structural elements and the frame, the type of wood, the interaction with other materials, and the connections among different elements of the frame.

A common timber frame wall constructive system is the half-timbered wall. This system is composed of timber elements (beams, posts and bracings) that are joint to create a frame that interacts with a masonry infill. Another common constructive system is composed of timber elements that appear embedded in masonry at the floor level and at different heights of the wall (Poletti, 2013). The interaction of timber and masonry causes the frame to be resistant to compressive stresses due to the masonry and to tensile stresses due to the timber. However, the existence of different variations, configurations, and typologies may result in complex structural responses that are interesting to study.

Many timber frame structures have historical and cultural significance that provide character to a particular region. For example, the Pombalino timber frame in Portugal, the Quincha timber frame in Peru, and the timber frame houses in Valparaiso, Chile (Figure 1.1). These and many timber frame buildings have a distinct architecture and construction technique that makes them a valuable architectural heritage.

In many regions these valuable structures seem in danger due to natural events like earthquakes or due to demolition. Even that timber frame structures have shown a good structural behavior toward seismic events, in most regions there is lack of knowledge about their seismic performance and safety becomes a concern. When this occurs, in some cases, the demolition of the building is considered. A solution, to prevent this issue and to prevent the loss of cultural heritage due to seismic events, is to evaluate the structural behavior of these buildings with the aim of ensuring safety and an efficient restoration and conservation. To obtain this aim the expertise of engineers, architects, historians and all conservation experts should be involved. Since there are limited studies about the structural response of timber frame buildings under seismic loading, this master thesis seeks to contribute to their study and conservation.

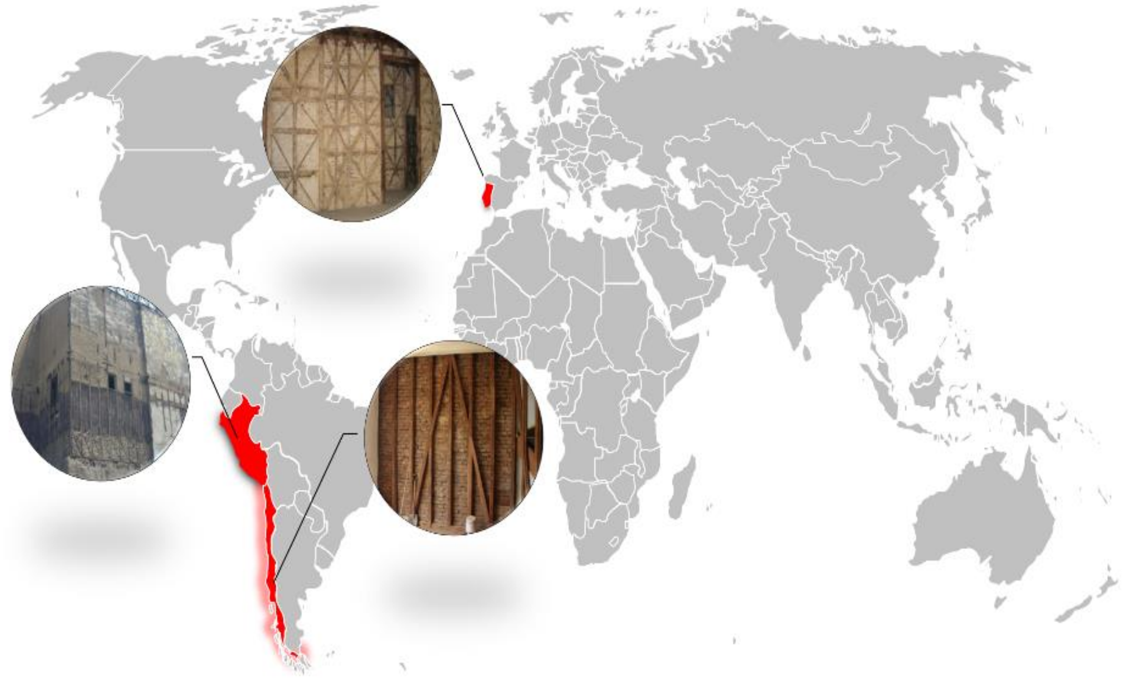

Figure 1.1. Historical timber frames around the world. Left to right: Quincha timber frame in Peru, Valparaiso timber frame in Chile, and Pombalino timber frame in Portugal.

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1.2 Objectives

The aim of this master thesis is to focus in the vulnerability analysis of timber frames in order to determine their seismic risk. Several vernacular timber frame typologies will be studied to determine their resistance and behavior toward seismic actions. To obtain this aim, numerical analysis of different structural configurations will be performed by including variations in material, geometry, bracings and carpentry joints types. The numerical analysis will provide the structural capacity of the frame, that will contribute to understand its global behavior and structural response.

The structural response of timber frames is mostly defined by the behavior of the connections. Since they are vernacular constructions, before the invention of nails, the most common joints were assemblies and the connections used are carpentry joints. These connections have a complex behavior that will also be studied in this master thesis. Previous experimental works and research in carpentry joints are considered and studied, since the numerical simulation of these connections may result challenging and complex. The final aim of this thesis is to perform a numerical analysis of a real case study that has not been analyzed before. To obtain this aim an appropriate calibration of the numerical model is performed by applying the numerical simulation of the carpentry joint connections and other relevant assumptions.

1.3 Methodology

The methodology used for this study includes:

- Literature Review

  The literature review consists of doing research about previous works. Specifically, about previous experimental and numerical investigation about timber frame structures.

- Choice of the Modelling Approach

  After the literature review the modeling method to be used is selected for the purpose of the research. The lumped plasticity model is selected since it requires a low computational effort, and it considers a concentrated non-linearity and the behavior at the connections.

- Numerical Model Calibration

  The model calibration requires experimental works to obtain information about the overall behavior of the frame and the carpentry connections. With this information, it is possible to create a detailed numerical model were the behavior of the connections is introduced as a spring or as a non-linear element. For this thesis study, previous experimental works of timber frames from different experimental campaigns were used as a reference. They were selected in order to obtain a good calibration for the behavior of the timber frame in a numerical model.

- Numerical Analysis

  After the numerical model is calibrated based in previous experiments, the next step is to create a numerical model of a real case study. Several numerical models were created starting from the elementary cell of the frame, then a shear wall and finally the modelling of two representative cases of timber frames. For all the models a nonlinear static analysis (pushover) was performed.

1.4 Outline of the Thesis

The work is organized in the following chapters:

Chapter 1 – Presents the introduction of the work, including the motivation, background, and the objectives of this thesis.

Chapter 2 – Presents a detailed literature review on timber frame structures, including available experimental studies and numerical approaches.

Chapter 3 – Provides a description of the calibration of the numerical models based on the use of beam elements, within the FEM theory, and with lumped plasticity. The FEM models are calibrated by comparison with experimental tests available in the literature.

Chapter 4 – Provides a description of the typical structural configurations of timber frames of the city of Valparaiso, Chile, by considering the geometry of the members, the type of wood and the carpentry connections.

Chapter 5 – Presents the numerical simulation of two representative real cases of timber frame shear walls of the city of Valparaiso, Chile.

Chapter 6 – Presents the final remarks of the research, the principal conclusions derived from the study and recommendations for future works.

CHAPTER 2 STATE OF THE ART

2.1 Timber Frame Structures: Historical and Technical Background

The origin of timber frame structures probably dates back to the Roman Empire, since evidence of a half-timber frame construction from ancient Rome refered to as *Opus Craticium*, was found in the town of Herculaneum by archaelogists (Poletti, 2013). Figure 2.1 shows a building from the town of Herculaneum that was uncovered by archeologists after it was buried in the eruption of Vesuvius. Timber was also used in previous cultures, since it was identified that masonry was reinforced using timber elements in the palaces at Knossos in Minoan Crete suggesting that timber-laced masonry construction dates to the 1500 to 2000 B.C. (Langenbach, 2015). In Greece, timber was mainly used to reinforce and repair buildings after they were affected by earthquakes, but then it became part of the new buildings constructions techniques.

Figure 2.1. c

Timber frame constructions spread througthout the world, first by having an apperance in Turkey in the eighth century. The expansion continued to other countries including Britain, were the construction system is refered to as “half-timber”, France were is refered to as *colombage*, and Germany were is refered to as *Fachwerk*. Portugal, Italy, Spain, Scandinavia and India are other countries to where timber frame construction technologies spread and became part of their vernacular architecture. The English, German and French developed their houses based in timber frames and due to immigration, they brought this constructive system to countries in North, South and Central America.

Due to the development and expansion of the timber frame construction around the world, is possible to find different configurations and typologies that share similarities in construction techniques and structural behavior. One of these similarities is their good structural behavior towards a seismic event. In many cases timber frame structures resulted to be more resistant than reinforced concrete buildings. For example, a collapsed reinforced concrete building next to a surviving timber frame building in Turkey after the 1999 earthquake is shown in Figure 2.2. A part of understanding the good behavior of timber frames, is to know the structural components that compose the frame. The general constitution of a timber frame constructive system is characterized by three main components:

1. **Bare frame**, composed of horizontal and vertical elements that resist the vertical load.
2. **Bracings**, mostly consisting of diagonal members that play a central role by resisting lateral forces.
3. **Infill**, composed of materials (stones, bricks, adobe, lath, plaster), that provide additional resistance.

Timber, which is the main component of the frame, is what provides a good resistance capacity against horizontal loads, due to the great elastic properties of the material. However, the lateral resistance of a timber frame is not only provided by the wood, since the infill and the bracings also play an important role. The materials used as infill usually have a good compression resistance, which improves the resistance of the frame. The materials that compose a timber frame have good mechanical properties that combined create a good resisting system. However, the geometry of the frame and the connections between the structural members turn to be relevant, since scientific research have validated that the location of the failure often coincides with the connections.

Figure 2.1. A collapsed reinforced concrete building next to a surviving timber frame building in Turkey after the 1999 earthquake. Photograph from: Adem Dogˇangün. (Langenbach, 2015).

In historical timber frames, the connections between timber elements are usually carpentry joints. The type and characteristics of theses joints depend on the geographical location, traditional techniques and the age they were created (Branco, 2015). Common traditional carpentry joints that can be found in historical timber frames include: mortise and tenon joints, notched joints, lap joints, and scarf joints.

Mortise and tenon joints are composed of two timber pieces, the mortise hole and the tenon tongue, that connect members that usually form an "L" or "T" type configuration, as shown in Figure 2.3(a). This kind of joint is mainly used when pieces connect at an angle between 45° to 90°, but when the angle is different from 90º, the nose of the tenon can be cut off and is called a skewed tenon (Branco, 2015), as shown in Figure 2.3(b).

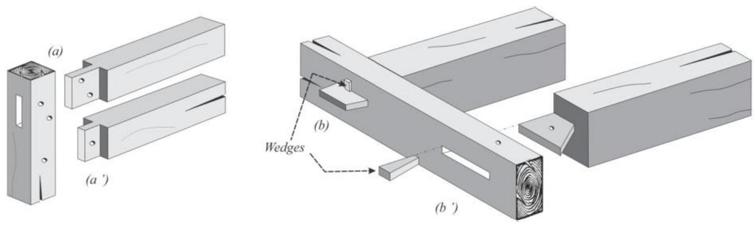


Figure 2.1. (a) Mortise and tenon joint. (b) Mortise and skeweed tenon. (Branco, 2015)

Notched joints consist of joining two pieces, by compressing one piece into another through a “V” shape groove usually perpendicular to the length of the element that is going to be connected to, as shown in Figure 2.4(a). A tenon can be added to the notched joint to keep all the beams coplanar, as shown in Figure 2.4(b), but the notch is what creates the strength of the joint (Branco, 2015).

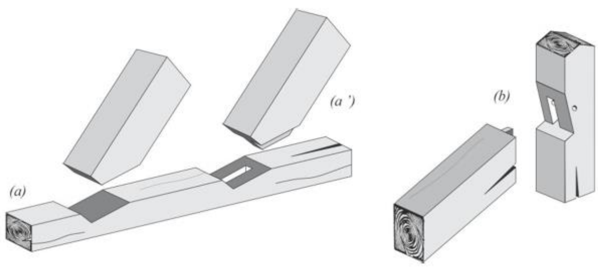


Figure 2.1. (a) Notched joint. (b) Notched joint with tenon. (Branco, 2015)

Lap joints can be classified in three types. One type is the full lap joint, were the joint is created without removing any material from the members that will be connected and is held in place by a pin, as shown in Figure 2.5(a). Another type is the half-lap joint, were the joint is created by removing material from both members, as shown in Figure 2.5(b). In this case usually half of the thickness of the members are removed. The last type is the dovetail-lap joint, were the joint is created by a tenon with a similar shape to the tail of a dove, as shown in Figure 2.5(c).

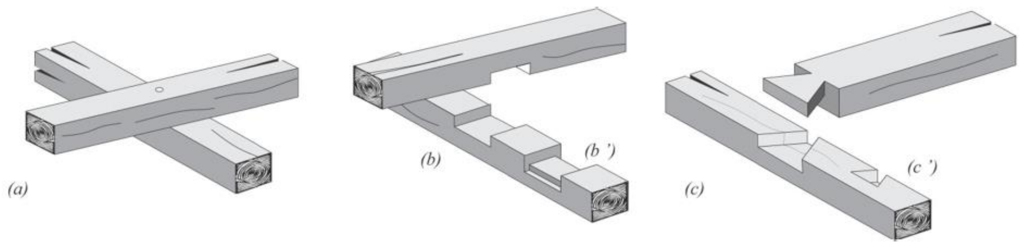


Figure 2.1. (a) Full lap joint. (b) Half-lap joint. (c) Dovetail-lap joint. (Branco, 2015)

Scarf joints are usually used when the required length of the members is not available, since this joint allows to connect members end to end, as shown in Figure 2.6. To secure the joint, usually pins are used. Scarf joints can be classified in different types based on their geometry, for example: (a) the halved-scarf joint, (a') the lapped dovetail scarf joint, (b) the scarf joint, (c) the scarf joint with under-squinted ends, and (d) the bolt of lightning joint, all shown in Figure 2.6.

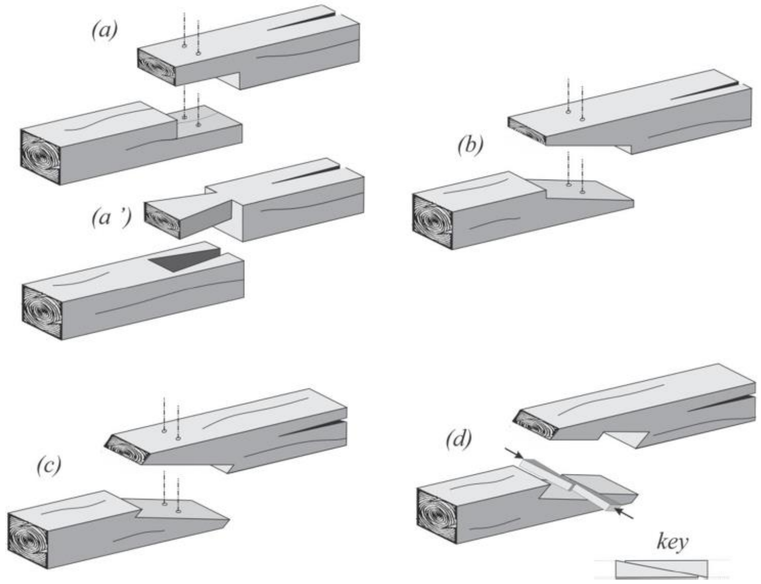


Figure 2.1. Scarf joints. (Branco, 2015)

The variation of the types of carpentry joints used, and other characteristics of timber frames like the geometry and materials created different typologies and constructive techniques over the years. Throughout history, timber frames resulted to be good and resistant structural systems. They spread throughout the world, especially in zones with seismic hazard. Today we can find interesting examples of historical timber frames, shown in Figure 2.7.

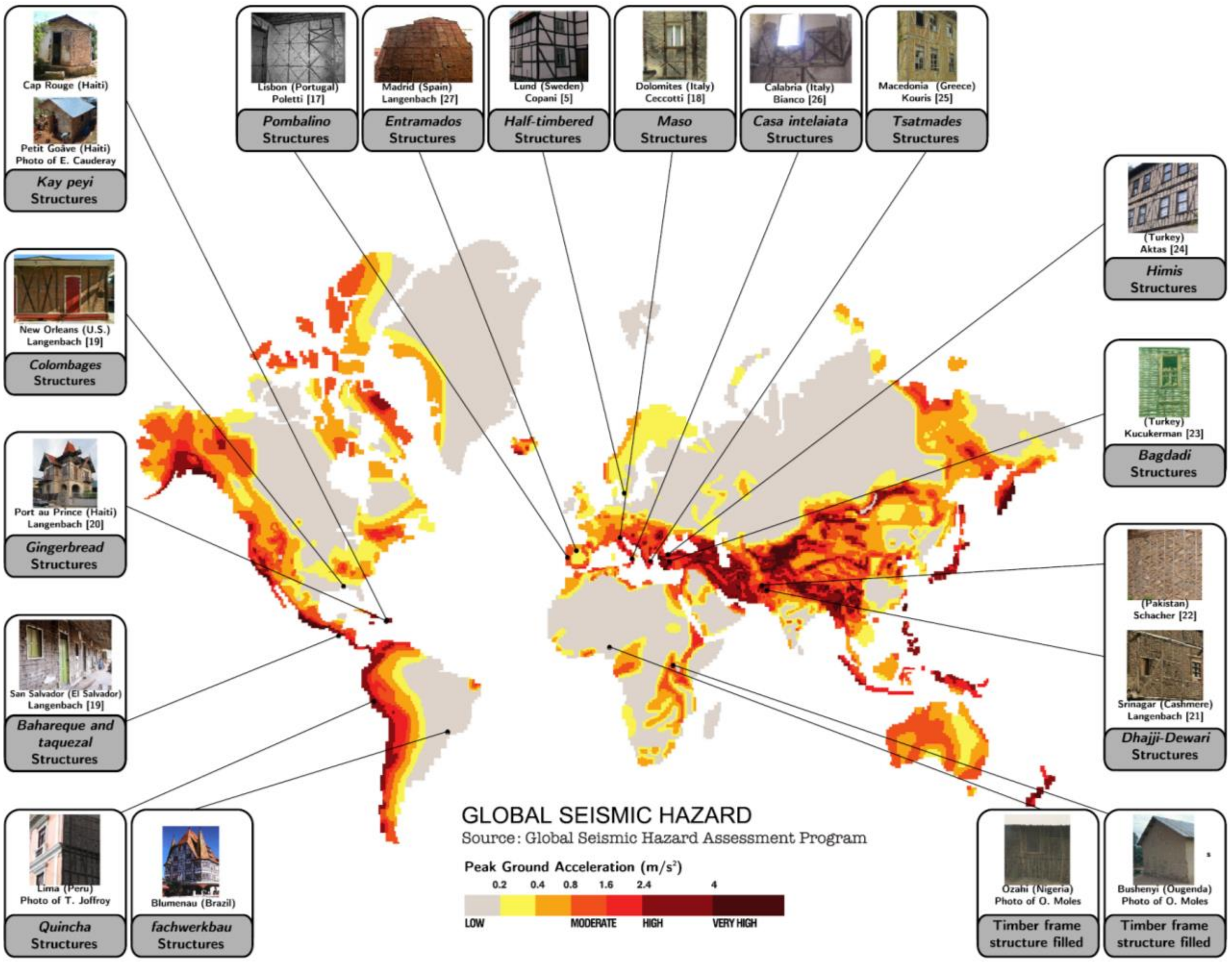


Figure 2.1 Timber frame structures around the world. Base map exhibits the global seismic hazard of each region. (Vieux-champagne et al., 2014)

A historical example of a timber frame building is the Gaiola. This construction typology was developed in Portugal after the 1755 Lisbon earthquake (Langenbach, 2015). Gaiola means cage and the name derived from the fact that the frame is composed of external masonry walls and an internal timber structure. This typology is also known as Pombalino since the Marquis of Pombal was the one that directed its development. The geometry of the frame consists of horizontal and vertical elements, and diagonal bracing members in an X shape, as shown in Figure 2.8. The usual carpentry joints between these elements are the connection by contact and the half-lap joints, shown in Figure 2.9. The infill is usually rubble or brick masonry, but it can also be composed of other materials like mud and hay, depending on their local availability.

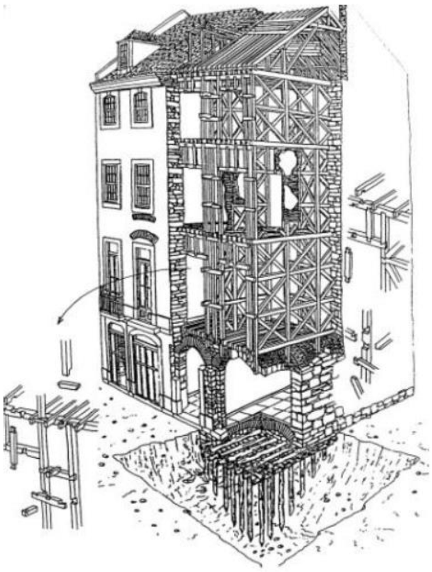


Figure 2.1. Pombalino building. (a) Structure of a whole building. (b) Example of a frontal wall. (Poletti, 2013)

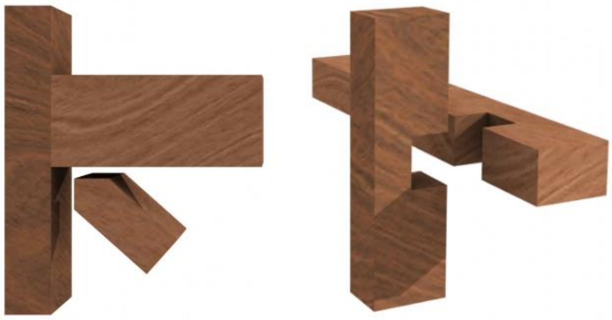


Figure 2.1 Carpentry joint connections: (a) Connection by contact. (b) Half-lap tee halving connection.

Another historical example of a timber frame building is *Casa Baraccata*. This typology was developed in Italy after the 1783 Calabria earthquake (Langenbach, 2015). The name comes from the term "baracca" which was the name provided to the temporary timber houses that were built for refugees after seismic events. This typology was developed based in the construction methods already used in Portugal. The timber frame composition is similar to Pombalino, with the difference that the pillars are not composed of a unique timber element. However, their position is constant from the foundation to the roof. The proposal for the construction of *Casa Baraccata* was by Giovanni Vivenzio and it consisted of a construction by blocks. In this case he proposed three blocks with the idea that the central building had a higher height and the lateral ones act as buttresses (Poletti, 2013). The timber frame was embedded in the external masonry to act as a reinforcement, as shown in Figure 2.10.

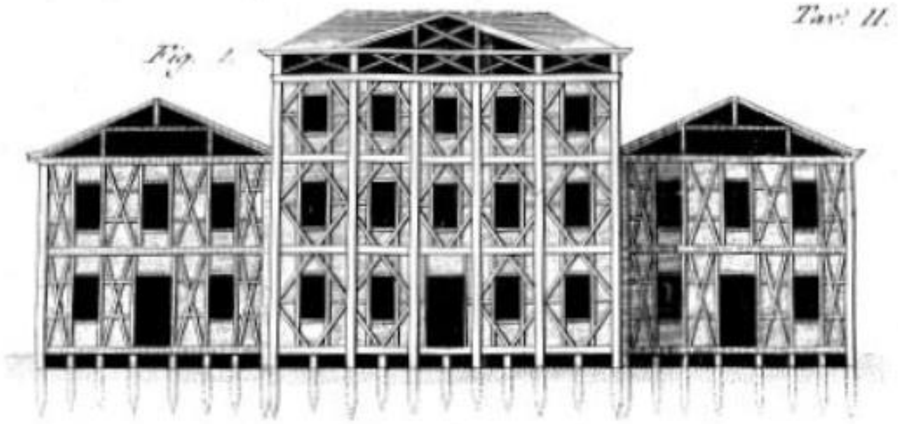


Figure 2.1 *Casa Baraccata* façade. (Poletti, 2013)

The Quincha timber frame is an example of a construction technique found in historic buildings in Central and South America. The word quincha is derived from the Quechua word “khincha” which means a wall of branches (Quinn, 2017). The Quincha wall is composed of timber elements that create a frame, which is infilled with woven cane and mud, as shown in Figure 2.11. The geometry of the bare frame consists of beams, posts, and diagonal bracings that are connected by carpentry joints, usually mortise and tenon joints, and lap joints, shown in Figure 2.12. This technique dates back to 2600 BC, since a primitive form was found in Peru in the archeological site of Caral (Quinn, 2017). Timber frames with an infill of canes and mud were used in Central and South America since Pre-Hispanic times and it developed to different variations. The technique was usually used for the construction of rural houses and they were referred to by different names including *bahareque* in Costa Rica, *pajareque* in Honduras, *vareque* in Ecuador, *pared francesa* in Argentina, and *cañizo* in Spain.

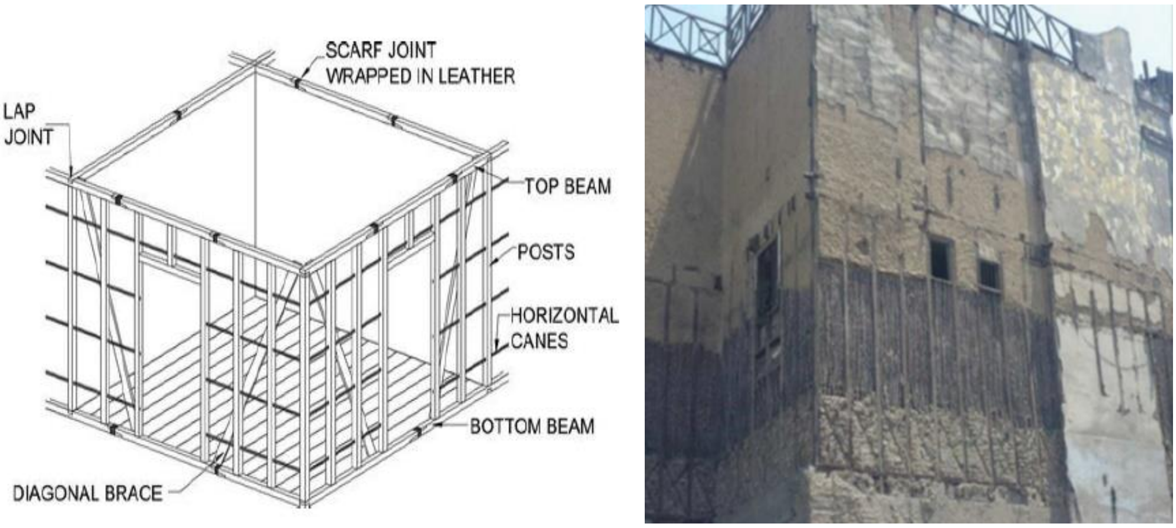


Figure 2.1. Quincha building: (a) Structure of a whole. (b) Example of a wall. (Quinn, 2017)

Figure 2.1 Carpentry joint connections: (a) Lap joint connection. (b) Mortise and tenon connection.

2.2 Experimental Campaigns on Testing Timber Frame Structures

Experimental works have been done previously to prove the effectiveness of historical timber frames when subjected to seismic loads. The experimental results have shown that the seismic response of the frame will depend of the wall geometry, the infill and type of connections. To study timber frame structures experimentally, they are subjected to monotonic and cyclic tests. Monotonic tests are performed by applying an increasing load to determine the ultimate capacity of the shear wall. While, cyclic tests are used to simulate the seismic action and obtain information about the shear resistance and global behavior of the system. These types of tests are useful to further develop analytical and numerical models that will contribute in the prediction of the seismic resistant behavior of these structures. Several experimental campaigns have studied real cases and replica specimens of infilled and bare timber frames typologies, like the following examples.

1- The CNR Ivalsa in Trento, Italy, carried out an experimental campaign on a full-scale specimen of the Mileto masonry reinforced with timber framing (Borbone constructive system), shown in Figure 2.13(a) (Ruggieri and Zinno, 2015). The experimental test included cyclic tests by applying load increments to two frames, one with infill and a bare frame.

2- The laboratory of Domaine University in France carried out an experimental campaign of a Haitian timbered masonry structure called ‘‘Kay peyi’’, shown in Figure 2.13(b) (Vieux-champagne et al., 2014). In this campaign the frame was studied in three scales: the connections, the elementary cell and the shear wall, as shown in Figure 2.14. The tests performed included cyclic and monotonic tests. The advantage of the approach used in this study is that it provides an understanding of the behavior of the components of the wall, which results useful for further numerical investigations.

Figure 2.2 (a) Borbone constructive system tested in CNR Ivalsa in Trento, Italy. (Ruggieri and Zinno, 2015) (b) Haitian timbered masonry structure tested in Domaine University in France. (Vieux-champagne et al., 2014)

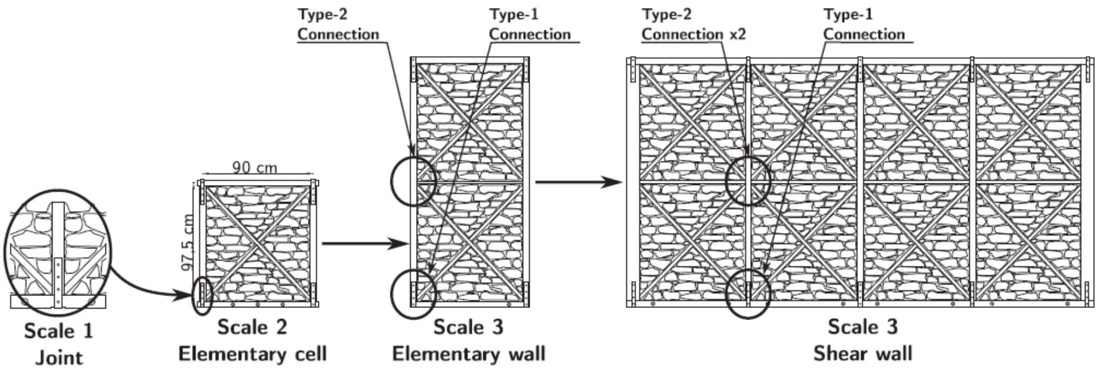


Figure 2.2. Three scale study of the Haitian timbered masonry structure. (Vieux-champagne et al., 2014)

3- The University of Minho in Portugal carried out an experimental campaign to study the seismic performance of traditional half-timbered walls (Pombalino) with a real scale wall specimen in the laboratory, as shown in Figure 2.15(a) (Poletti, 2013). The experiment included cyclic tests for both, an infill and a timber frame specimen. The experimental procedure also included testing of the traditional timber connections.

4- The Structural Engineering Laboratory of the University of Bath in UK carried out an experimental campaign to study a half-scale timber frame of the Quincha frame from Peru, as shown in Figure 2.15(b) (Quinn, 2017). The tests performed included a three-phase loading protocol: Phase 1: a stabilizing load cycle, Phase 2: a stiffness load cycle and Phase 3: a strength test. The tests were performed for a timber frame and for a frame with infill. The campaign also included the testing of traditional mortise and tenon connections in order to study their nonlinear behavior.

Figure 2.2. (a) Pombalino half-timbered frame tested in the University of Minho in Portugal. (Poletti, 2013) (b-c) Quincha timber frame tested in the University of Bath in UK. (Quinn, 2017)

The next sections will discuss in detail the experimental campaigns of Pombalino (Poletti, 2013) and Quincha (Quinn, 2017), since in this thesis they will further be used to validate the calibration of the numerical model of a real case study that has not been studied before.

2.2.1 Pombalino Timber Frame Experimental Campaign

To study the seismic response of traditional Portuguese timber frame walls, quasi-static in-plane cyclic tests were performed on real scale specimens (Poletti, 2013). The frame tested in the University of Minho in Portugal, was constructed using the common dimensions of existing buildings with a total width of 2420 mm and a total height of 2360 mm, as shown in Figure 2.16. The frame was composed of four braced cells with dimensions of 840 x 860 mm^2. The top and the bottom beams had a cross section of 160 x 120 mm^2, while all the other elements had a cross section of 80 x 120 mm^2. The material of the frame was Maritime Pine (Pinus Pinaster), characterized by the properties shown in Table 2.1. The connections between the elements of the main frame are half-lap joints, while the diagonals have a connection by contact with a nail.

Table 2.1. Material Characteristics of Maritime Pine. (Poletti, 2013).

$f_{t,0}$	15	MPa	G_v	700	MPa	γ	9	-
$f_{t,90}$	5	MPa	v	0.3		$G_{ft,0}$	70	Nmm/mm^2
E_0	11000	MPa	ρ	590	kg/m^3	$G_{ft,90}$	50	Nmm/mm^2
E_{90}	5000	MPa	α_T	1	-	$G_{fc,0}$	130	Nmm/mm^2
$f_{c,0}$	25	MPa	α_h	1	-	$G_{fc,90}$	70	Nmm/mm^2
$f_{c,90}$	3	MPa	β	-1	-	k_p	0.001	-

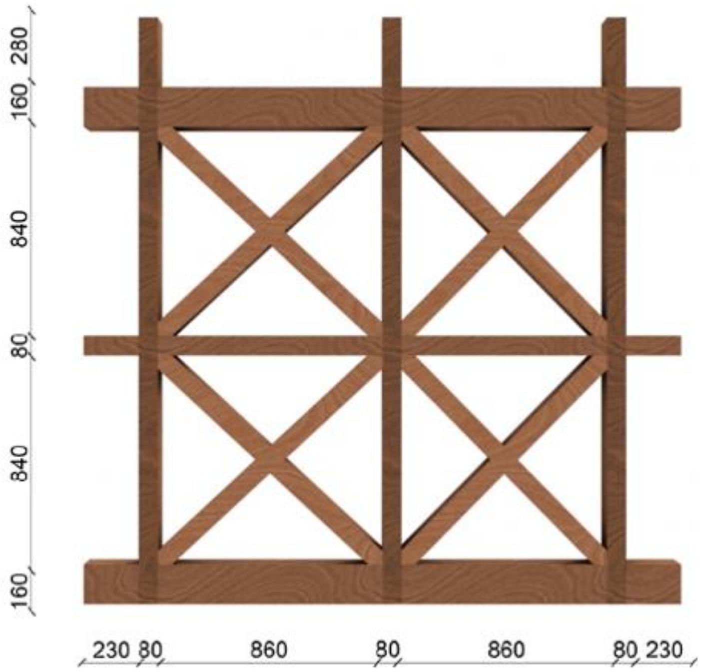


Figure 2.2.1 Geometry of the Pombalino timber frame. Dimensions in mm.

The experiment consisted of the following tests performed by applying the procedure stated by ISO 21581 (2010):

- Preliminary monotonic tests to validate the prevention of the out of plane movement and to calculate the displacement capacity of the wall.
- Quasi–static in–plane cyclic test:
    - Step 1: Pre-application of vertical load by applying 25 kN and 50 kN at the location were the beam and post intersect.
    - Step 2: Cyclic application of a horizontal displacement history on the top beam.

The test setup was arranged by setting the frame specimen on top of a steel profile that was connected to the three posts and to the reaction floor. The steel profile and the bottom beam were connected in several points to prevent the movement of the beam. Two steel plates were used at the top beam to apply the cyclic displacement at the top of the wall. Also, punctual steel rollers were used at the top beam to avoid out-of-plane displacements. Vertical hydraulic actuators were used to directly apply the vertical load at the three posts of the wall and a hydraulic servo-actuator was used to apply the horizontal displacement at the top beam. The global and local behaviors of the wall were captured by linear voltage displacements transducers (LVDTs).

The results of the tests, in general, showed that the wall has a good capacity and ductility. However, damages were concentrated at the connections. The timber frame, tested as a cantilever, resulted in a shear resisting mechanism as shown in the force-displacement hysteresis diagrams in Figure 2.17. The test showed that when a higher pre-compression vertical load was applied, the load capacity and the stiffness were higher. For the case of a higher pre-compression load, failure occurred at a 70.8 mm displacement; while for the case of lower pre-compression load, failure occurred at 30.4 mm displacement.

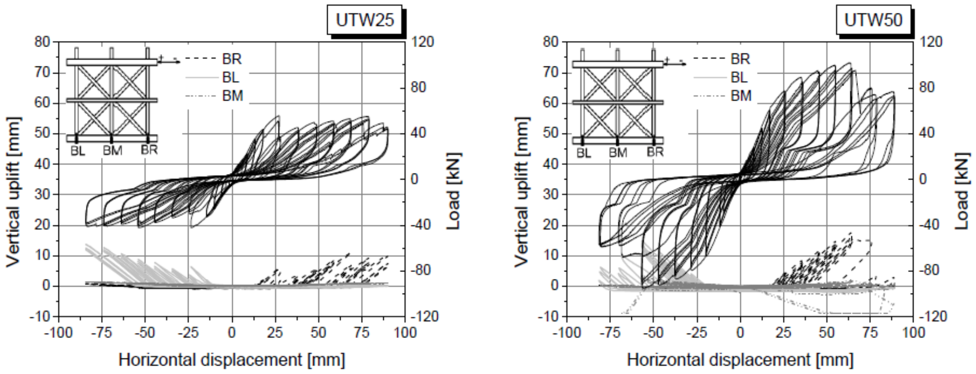


Figure 2.2.1 Force-displacement hysteresis diagrams for Pombalino timber frame. (Poletti, 2013).

The response of the timber frame was mainly in shear, characterized by the bending of the posts, as shown in Figure 2.18. Regarding the behavior of the connections, Poletti (2013) observed that in the connection by contact between the post and the diagonal, they separated when forces were applied along the diagonals, as shown in Figure 2.19(a). Another observation was that after failure, the diagonals started to open with a higher rate and all the connections moved freely, as shown in Figure 2.19(b). In the case of the half-lap connections there was a concentration of damage in the central connection, since it crushed due to the lateral compression applied by the diagonals, as shown in Figure 2.19(c).

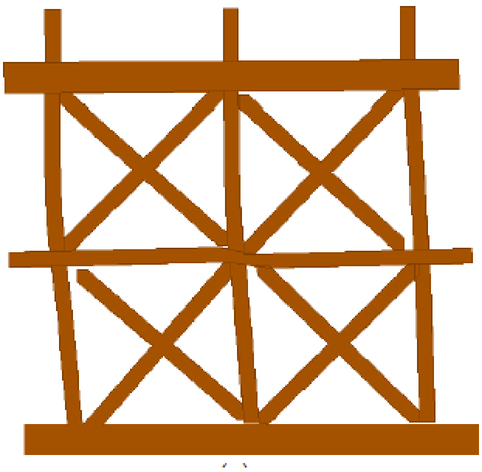


Figure 2.2.1 Behavior of the wall during the test. (Poletti, 2013)

Figure 2.2.1 Damage of the connections: (a) opening of lateral connection. (b) opening of diagonals. (c) crushing of central connection. (Poletti, 2013)

2.2.2 Quincha Timber Frame Experimental Campaign

To study the seismic response of traditional Quincha timber frame walls from Peru, in-plane tests were performed for half scale specimens (Quinn, 2017). The specimen tested in the laboratory of the University of Bath in UK was a half scale timber frame with a total width of 1190 m and the total height of 1690 m, as shown in Figure 2.20. The frame was composed of three bays with dimensions of 1690 x 357 mm^2. The beams and posts had a cross section of 60 x 80 mm^2, while the diagonal had a cross section of 90 x 30 mm^2. The material of the frame was Cypress, except for the low beam that was Sapelli, characterized by the properties shown in Table 2.2. The connections between the beams and posts are mortise and tenon joints, while the diagonals have a lap joint with nails.

Table 2.2.2 Material Characteristics of Sapelli and Cypress. (Quinn, 2017).

Wood	Sapelli	Cypress
E (kN/m^2)	7.7E+06	6.4E+06
ρ (kg/m^3)	400	390
v	0.3	0.3

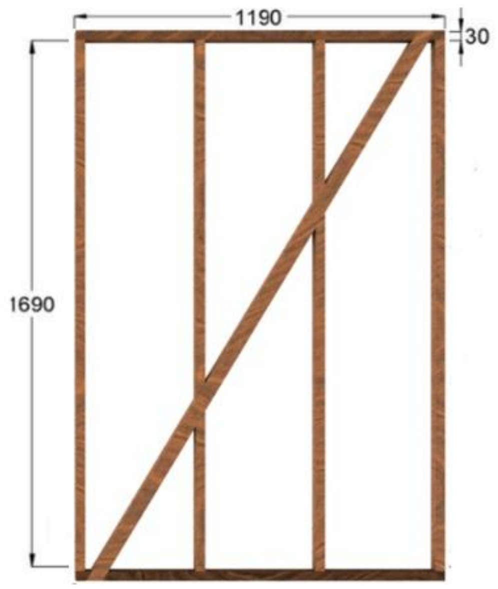


Figure 2.2.2 Geometry of the Quincha timber frame. Dimensions in mm. (Quinn, 2017).

The experiment consisted of in-plane tests performed by following the procedure stated by BS EN 594 (2011). The frame was tested in two directions, with the diagonal in tension and in compression, because it is asymmetrical and reverse cyclic loading was not feasible. As recommended by BS EN 594 (2011), three-phase loading protocol was selected for the tests, as shown in Figure 2.21:

- Phase 1: Stabilizing load cycle
  The cycle remained with a load of 0.2 kN applied at a rate of 40 N per second for a period of 120s to allow the joints to settle in, and then a period of 300s recovery.
- Phase 2: Stiffness load cycle
  A loaded controlled test with 3 stiffness cycles, with increasing loads of 0.5kN, 0.7kN and 0.9kN.
- Phase 3: Strength test
  The frame is loaded until failure.

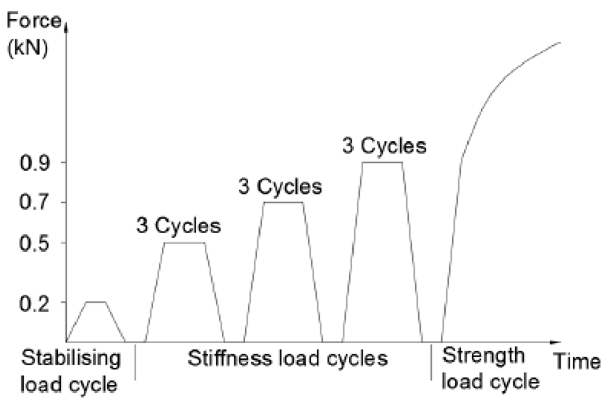


Figure 2.2.2 Horizontal loading protocol for UK tests. (Quinn, 2017).

The test setup was arranged by fixing the base of the frame specimen to the strong floor by steel plates with bolts. The purpose was to have a fixed condition to prevent sliding. Two hydraulic jacks linked to the same hand pump were used to apply the vertical load and a hydraulic jack connected to a load actuator was used to apply the horizontal load at the top beam. A steel spreader beam was attached to the vertical load cells and a steel plate was fixed to the top beam. To ensure an even distribution of the load, rollers were inserted between the two steel sections. Steel reinforcing bars were used in a vertical position on each side of the frame to prevent out-of-plane movement. To observe the deflected shape of the external posts several transducers were used. Figure 2.22 shows the setup of the frame.

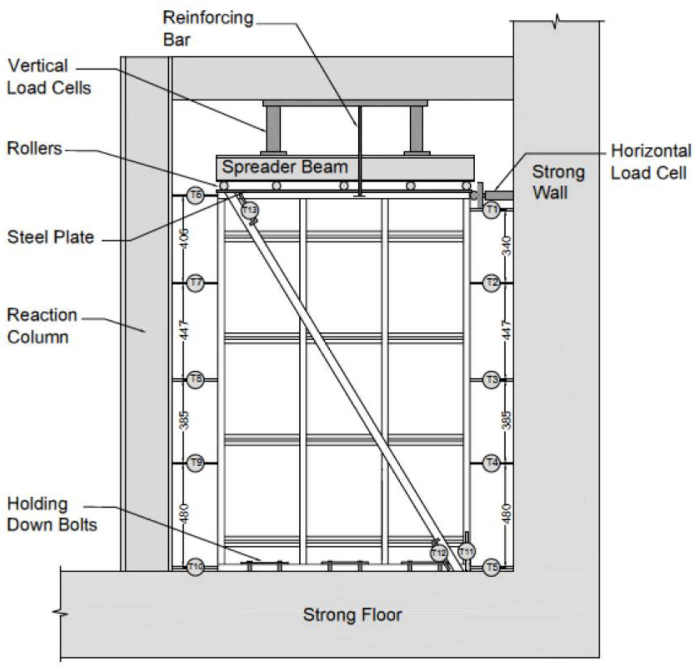


Figure 2.2.2 Quincha timber frame setup for testing. (Quinn, 2017)

The results of the stiffness test show that the frame is mostly within the elastic range, as seen in Figure 2.23(a). Negligible lateral deformation occurred and no uplift of tenons was observed during this test. During the strength test once the diagonal started to yield the capacity of the frame continued to reduce, as shown in Figure 2.23(b). The observed deformation of the frame was that the post closest to the applied load was deforming in a cantilevering shape showing relative rotation between the beam and post, while the external post on the opposite side deformed in an S-shape (Quinn, 2017), as shown in Figure 2.24. Also, the internal beam-post connections rotate more than the outer ones due to the influence of the diagonal (Quinn, 2017). However, the diagonal showed some bending, since it acts as a tie but also has moment capacity due to the nailed connections. The ultimate failure of the frame occurred when the diagonal failed in a combination of direct tension and in-plane bending (Quinn, 2017).

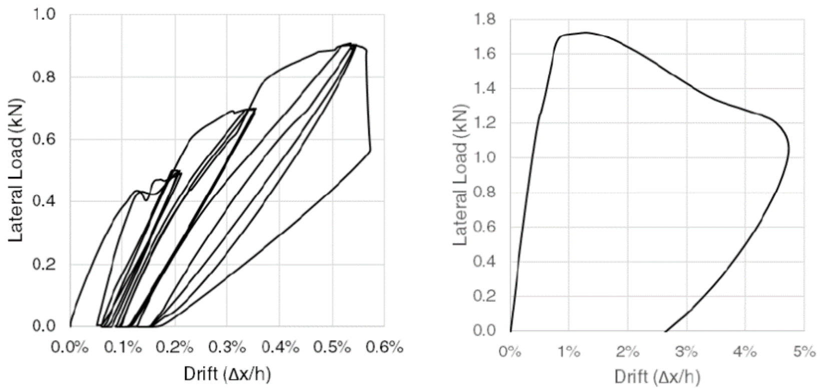


Figure 2.2.2 (a) Stiffness test. (b) Strength test. (Quinn, 2017).

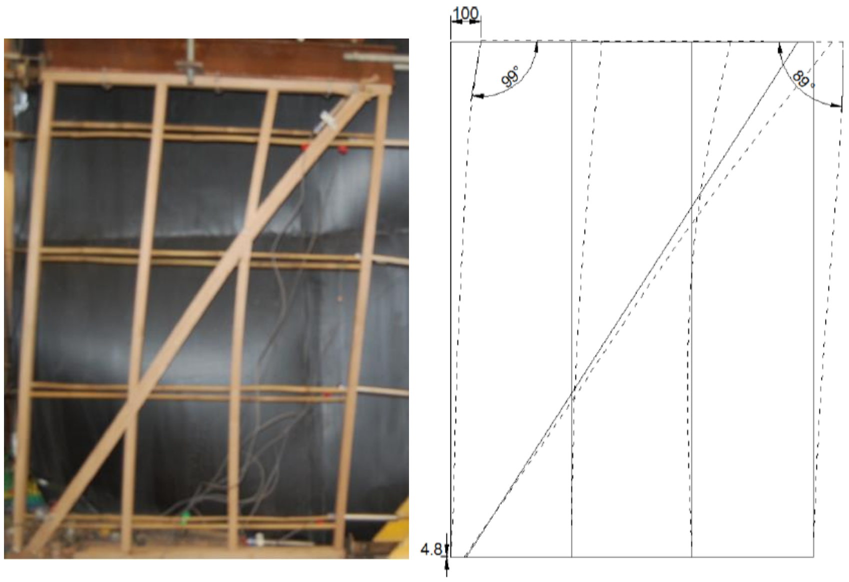


Figure 2.2.2 Deflected shape of the timber frame. (Quinn, 2017).

Quinn also studied the behavior of the connections and their characteristics. For the mortise and tenon connection, Quinn (2017) studied its rotational and translational stiffness. To determine the rotational stiffness a separate specimen of the connection was tested. The specimen was manufactured with a 60 mm square post and a 25 mm square tenon with 50 mm of length, as shown in Figure 2.25(a). The connection was tested by pushing the post horizontally inducing a moment until failure. Two specimens were tested, one with a tighter fit than the other. The obtained results are shown in Figure 2.25(b).

Quinn (2017) also studied the possibility of determining the rotational stiffness based in the geometrical properties of the connection. To obtain a relation between geometry and rotational stiffness the component method was applied but modified depending on the center of rotation of the connection. The center of rotation marked with an X, in Figure 2.26(a), was determined by assuming that only the two portions of the tenon that are directly in contact with the walls of the mortise contribute to the stiffness. Using this assumption to modify the component method Quinn determined a realistic estimate to relate the length of the tenon and the rotational stiffness, as is shown in Figure 2.26(b).

To determine the translational stiffness of the mortise and tenon connection a set of localized tests are required but were not included. Since there is no dowel to give tensile capacity to the mortice and tenon joint, a defining feature of the frames is the ability of the posts to move upwards (Quinn, 2017). Due to this behavior, Quinn (2017) assumed the translational stiffness based in the experimental data of vertical uplift measured for the frame, as shown in Figure 2.27.

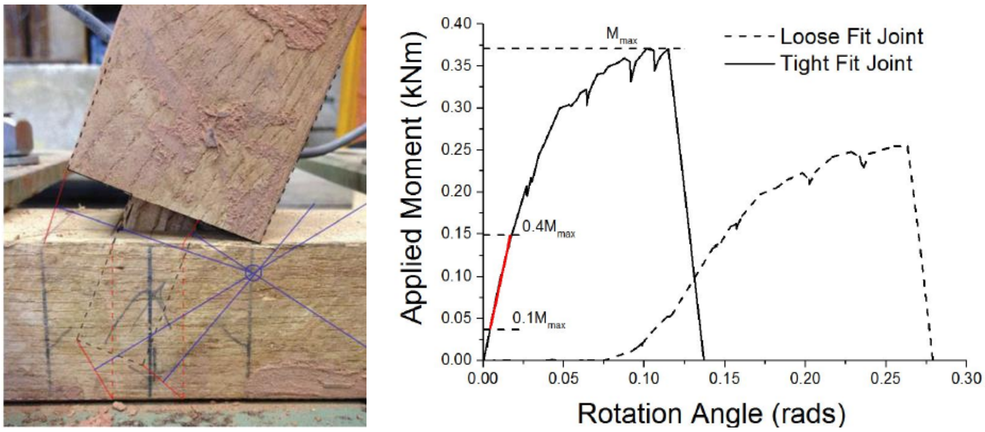


Figure 2.2.2 (a) Mortise and tenon tested. (b) Moment rotation curve. (Quinn, 2017).

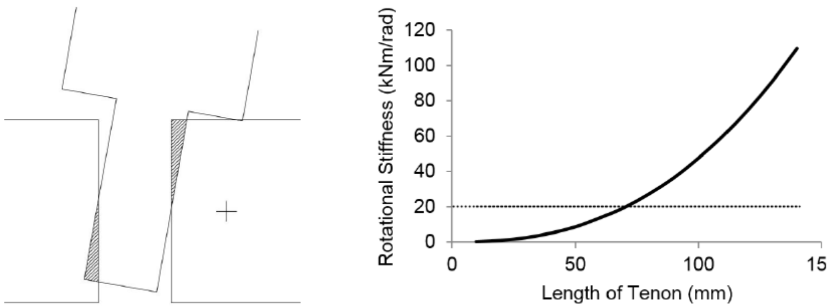


Figure 2.2.2 (a) Diagram showing portions of tenon contributing to stiffness. The cross marks the assumed center of rotation. (b) Variation in stiffness with length of tenon. (Quinn, 2017).

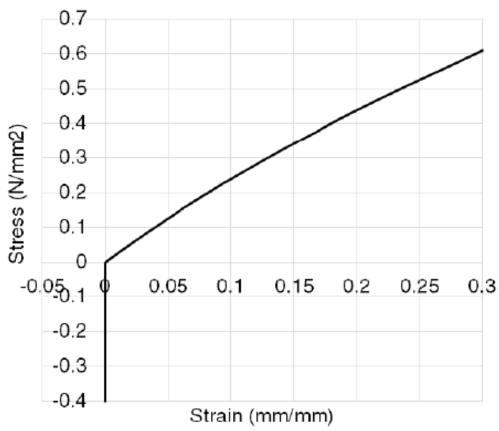


Figure 2.2.2 Stress-strain relationship for tenons. (Quinn, 2017).

2.3 Numerical Modelling of Timber Frame Structures

The Finite Element Method (FEM) is a numerical tool useful in structural engineering and one of the most used approaches to perform numerical models of structures. Finite element analysis can be applied to simulate the behavior of an entire structure or of a single structural component. The complexity of the finite element model will depend on the type of study, considering the discretization of the physical object modeled and the division of the interconnected elements.

A numerical model for a timber frame structure must consider the geometry of the frame, the material properties of the structural members, the loading and boundary conditions, and the behavior of connections. However, the method of analysis will depend on the purpose of the analysis and the expected output. Two main modelling approaches are the micro level model and the macro level model. Micro-models entail a high detail of analysis, but they require several input parameters and a high computational effort. On the other hand, macro-models represent a simplified approach that requires less input data and more affordable computations. Hybrid models are also a possibility when, for example, the purpose is to analyze in detail a specific structural element within a more complex structure (Costa et al., 2014). These considerations are important to select the best approach to be used in the development of a numerical model of a specific structural problem.

2.3.1 Micro Model

A micro model is a small-scale model that provides a local behavior of the structure with a detailed level of discretization for all its components. This modelling approach is useful to determine failure mechanisms, but it requires a high computational effort. It is also useful to simulate separately the behavior of a basic component of the structure. The micro-modelling strategy can provide results with a high level of accuracy, but the amount of time needed to obtain the input parameters and the modelling computational effort causes this method to become less efficient in the application to some cases.

An example of the application of micro modelling to timber frame structures was the model developed by Doudoumis (2010). The model consisted of a composite timber-masonry wall, as shown in Figure 2.28. In this case the behavior of the timber frame was simulated by inelastic elements with an elastoplastic behavior, and the masonry infill by shell elements with a pressure dependent material strength. The behavior of the connections was modeled by elastoplastic link elements and the interaction between the timber frame and the masonry infill by appliying the Coulomb’s law. Another application of micro modelling is the example of the model developed by Costa et al. (2014). This case consists of the anlaysis of a stone masonry structure from Azores, for which the masonry is reduced to its basic components: joints, blocks, and infill, as shown in Figure 2.29. The model assumed continum finite elements for the units and the mortar at the joints, and their interface was modeled by descontinuous elements that account for potential crack or slip. (Costa et al., 2014).

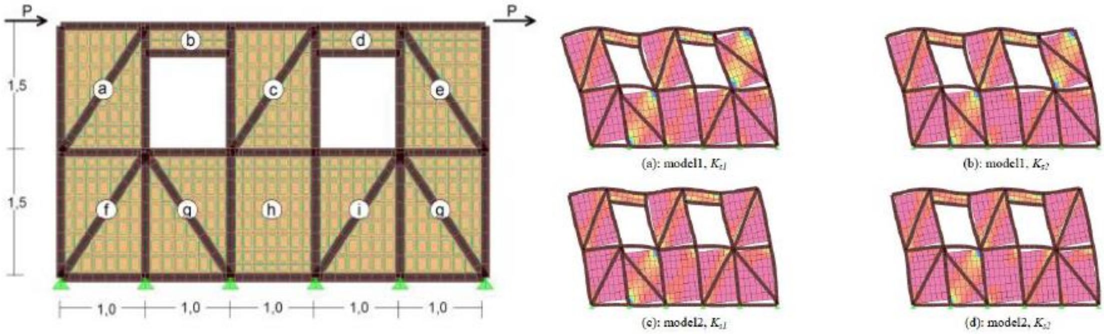


Figure 2.3.1 Micro model of a composite timber-masonry wall. (Doudoumis, 2010).

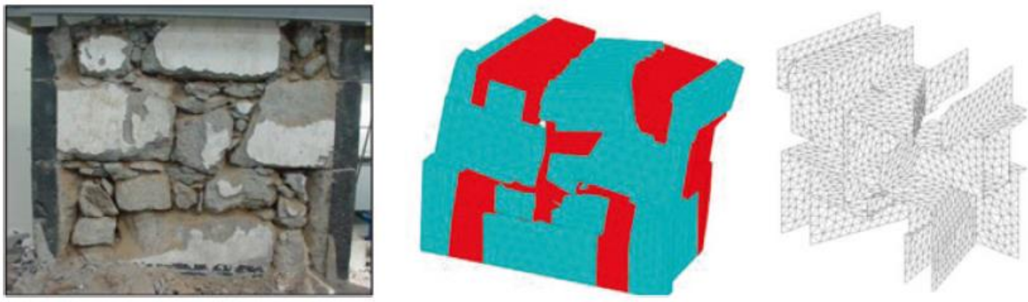


Figure 2.3.1 Micro-modelling of a masonry wall from Azores. (Costa et al., 2014).

2.3.2 Macro Model

A macro model is a large-scale model that provides an overall response of the structure. This modelling approach results to be simple and with a low computational effort, providing a good balance between simplicity and accuracy. Due to this, macro models are commonly used to characterize the seismic structural response of historical buildings. For example, they are useful to estimate the lateral load capacity of timber frame buildings.

A macro model can be applied by using a lumped plasticity or a distributed plasticity modelling approach. In the lumped plasticity model, nonlinear behaviors are assumed at the extremities of the structural element, while the body is modeled as an elastic part (Rahai and Nafari, 2013). However, the calibration of the inelastic element parameters will define the accuracy of the results. The distributed plasticity model assumes the nonlinear behaviors to occur at any element section. In this case the behavior of the section can be described in accordance with either fiber modeling approach or response curves reproducing the element behavior under reversible load (Rahai and Nafari, 2013).

Several lumped plasticity models have been developed. For example, Lukic et al. (2018) developed a numerical model of a timber frame wall by first modelling the behavior of the connections based on an individual connection test, and then applying the calibrated nonlinearity to the modelling of the timber frame wall. The procedure consisted of calibrating the joints individually, as shown in Figure 2.30(a).

Then a two-dimensional numerical model of a timber frame wall was developed by applying the nonlinearities to the joints, as shown in Figure 2.30(b). The numerical model included beam-column elements for the horizontal and vertical members, truss elements for the diagonal bracings, and calibrated springs for the joints, which will control the global behavior of the frame. Ceccotti and Sandhaas (2010) also developed a numerical model based on the lumped plasticity modelling approach. In this case it was applied to a three-dimensional model of a X-Lam building, as shown in Figure 2.31. A similar procedure to the last case was used, but shear walls with X-Lam panels cannot be modelled with rotational springs, only with translational springs (Ceccotti and Sandhaas, 2010). The calibration of the springs was based in the results obtained from cyclic tests.

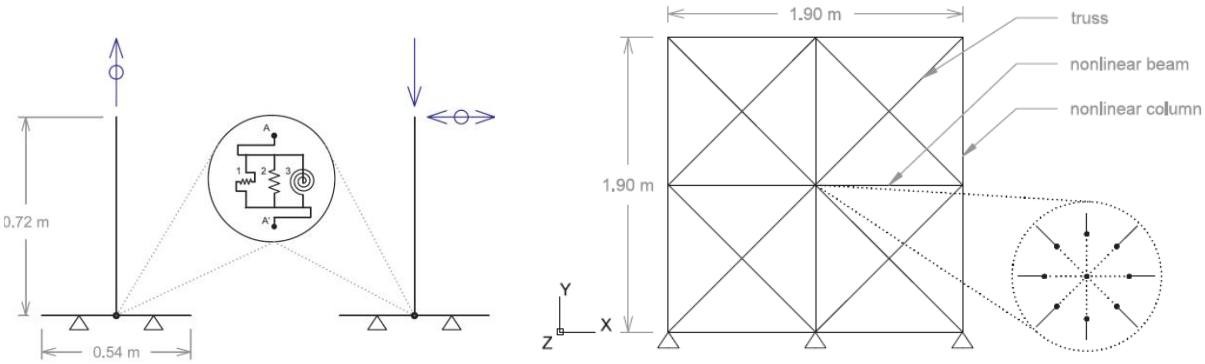


Figure 2.3.2 (a) Half-lap connection with three behaviors (1 – shear, 2- axial, 3- rotational). (b) Numerical model with link elements. (Lukic et al., 2018)

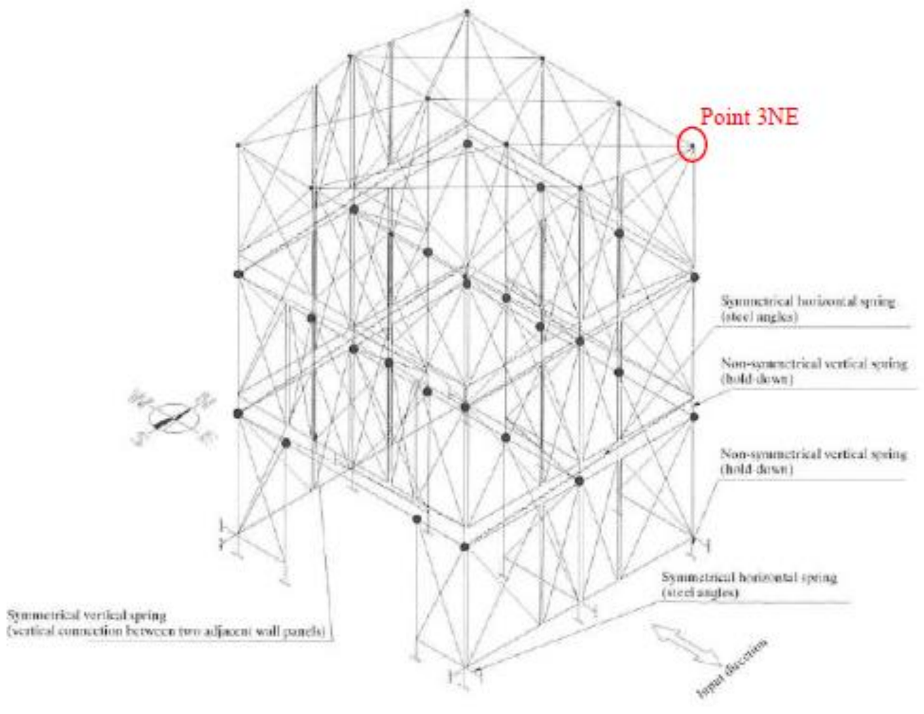


Figure 2.3.2 *Numerical 3D model of X-Lam building.* (Ceccotti and Sandhaas, 2010)

CHAPTER 3 CALIBRATION OF NUMERICAL MODEL

Considering the modeling approach of lumped plasticity described in Section 2.3.2, this chapter will explain the calibration process for two timber frame models that were previously studied by different experimental campaigns: Pombalino (Poletti, 2013) and Quincha (Quinn, 2017). The objective is to develop a numerical modeling approach that can be applied to similar typologies, since it is not always possible to obtain experimental results from a real case study. The model to be develop should be calibrated based on existing experimental results, reason why these previous studies will be considered.

3.1 Pombalino Timber Frame Calibration

This first calibration consists in simulating the mechanical behavior of the Pombalino timber frame reported in the experimental work by Poletti (2013), and the analytical model done by Ciocci (2015). The timber frame wall is modeled by using the finite element analysis softwares DIANA FEA and SAP2000. To validate the accuracy of the softwares and the calibration, the numerical capacity curves obtained by both models are compared with the experimental capacity curve. Finally, the experimental collapse mechanism of the frame is studied and compared those provided by both softwares.

The two-dimensional model considers the frame geometry, material properties, loading and boundary conditions. The geometry of the timber frame is composed of the elementary cell shown in Figure 3.1(a), with the dimensions described in Table 3.1. The material of the frame is Maritime Pine, modeled as a linear elastic isotropic homogeneous material with the properties described in Table 3.2. The loading conditions include a vertical load of 25 kN applied downwards at the three nodes were the top beam and the post intersect. An horizontal displacement of 0.1 m is applied at the left corner of the top beam. The boundary conditions include a restriction in horizontal and vertical directions at the nodes of the base. Also, a restriction in the horizontal direction is added at the node where the prescribed deformation is applied, as shownin Figure 3.1(b). For both modelling softwares, linear elastic beam elements are considered to represent the timber frame. However, the nonlinear behavior and finite stiffness of the carpentry joints is modeled by applying springs elements in DIANA FEA and concentrated hinges in

SAP2000. The carpentry joints of the Pombalino timber frame include: connection by contact and half-lap tee halving connection, as described in Section 2.1.

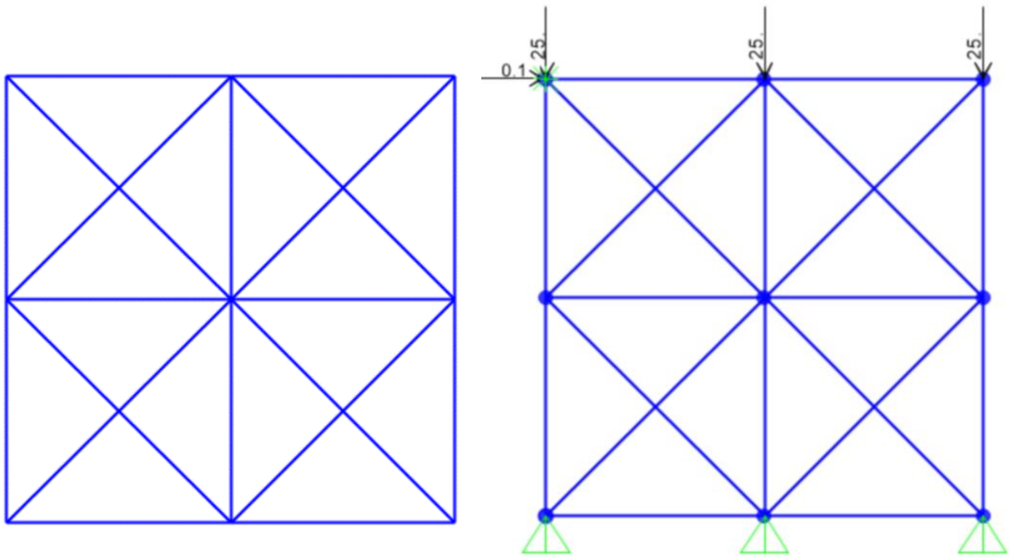


Figure 3.1. Elementary cell of the Pombalino timber frame: (a) Geometry. (b) Loading and boundary conditions.

Table 3.1 Cross sectional dimensions of the elements of the frame. (Poletti, 2013)

Elements	**Height (mm)**	**Width (mm)**
Beams	160	120
Posts	80	120
Diagonals	80	120

Table 3.1 Material Characteristics of Maritime Pine. (Poletti, 2013)

$f_{t,0}$	15	MPa	G_v	700	MPa	γ	9	-
$f_{t,90}$	5	MPa	v	0.3		$G_{ft,0}$	70	Nmm/mm^2
E_0	11000	MPa	ρ	590	kg/m^3	$G_{ft,90}$	50	Nmm/mm^2
E_{90}	5000	MPa	α_T	1	-	$G_{fc,0}$	130	Nmm/mm^2
$f_{c,0}$	25	MPa	α_h	1	-	$G_{fc,90}$	70	Nmm/mm^2
$f_{c,90}$	3	MPa	β	-1	-	k_p	0.001	-

The first step of the calibration is to analyze a rigid model and then create other models by adding the nonlinear behavior of the connections, as shown in Figure 3.2. The models were analyzed by:

- MOD 0 – Rigid connections
- MOD 1 – Hinged connections
- MOD 2 – Semi-rigid connections between the diagonal elements and the main frame
- MOD 3 – Semi-rigid connections between the diagonal elements and the main frame, and betweem the elements of the main frame

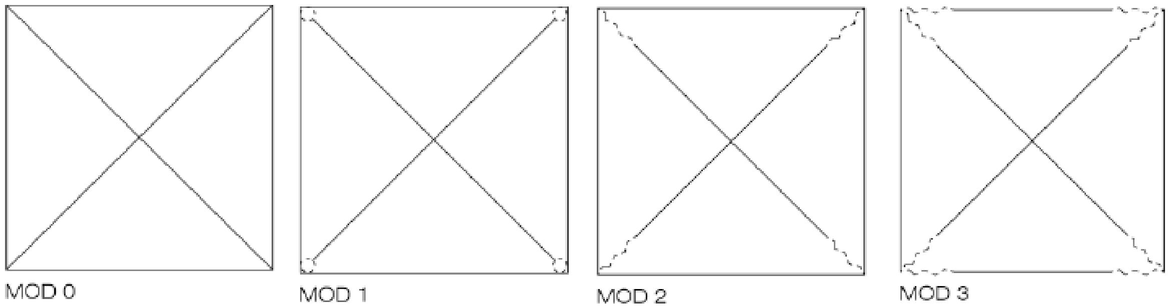


Figure 3.1. Typical cell for each numerical model. (Ciocci, 2015).

The first model analyzed is MOD 0 which considers all the connections as perfectly rigid. The second, MOD 1, considers a hinged connection between the diagonal and the frame by releasing all the rotational DOFs of each diagonal element at one end. The next models, MOD 2 and MOD 3, consider the connections as semi-rigid by introducing a stiffness value at the DOFs.

The behaviors of the semi-rigid connections are introduced in the model by considering force-deformation diagrams. The behavior of the connections by contact between the diagonals and the main frame are represented with axial and shear stiffness. While, the behavior of the half-lap connections between beams and posts of the main frame are represented with rotational stiffness. Ciocci (2015) developed an analytical model to compute the stiffnesses of the joints by applying the component method suggested by Drdácký et al. (1999), and Descamps (2009). The axial and shear stiffness of the connection by contact were calculated by applying the contribution of two stiffnesses: the stiffness provided by the contact area (k_c) and the stiffness provided by the nail, as shown in Figure 3.3. The stiffness provided by the contact area was calculated by considering two contact areas, as shown in Figure 3.3(a) and (b); obtaining the values shown in Table 3.3. The stiffness provided by the nail was calculated based on the Eurocode equation for nails without pre–drilling by considering the extraction stiffness (k_e) and the ultimate shear plane stiffness (k_{sp}), as shown in Figure 3.3(c); obtaining the values shown in Table 3.4. The rotational stiffness of the half-lap connection was also calculated by the application of the component method. In this case four contact areas were considered, as shown in

Figure 3.4(a). After performing the calculations of the component method the values shown in Table 3.5 were obtained by Ciocci (2015).

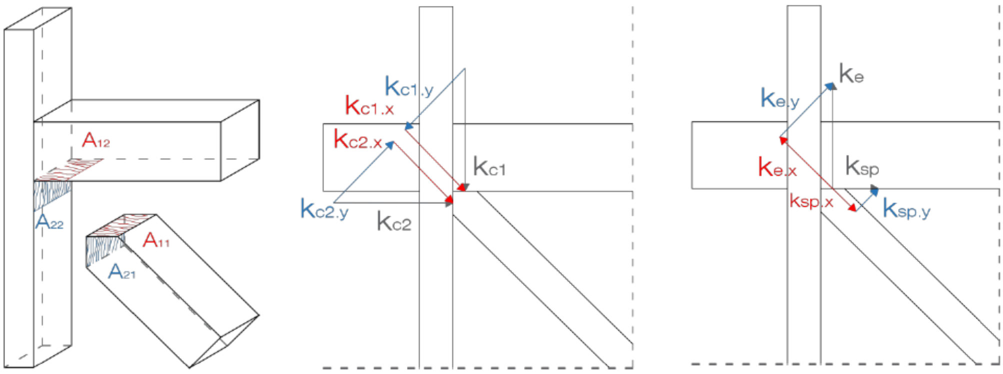


Figure 3.1 (a) Contact areas of the connection by contact, (b-c) Axial and shear stiffness for the connection between the diagonals and the main frame. (Ciocci, 2015).

Table 3.1 Stiffness K_c provided by the contact areas. (Ciocci, 2015).

A_1 (m^2)	**K_{c11} (kN/m)**	**K_{c12} (kN/m)**	**K_{c1} (kN/m)**	**K_{c1x} (kN/m)**	**K_{c1y} (kN/m)**
6.84×10^{-4}	1.15×10^{5}	6.20×10^{4}	4.02×10^{4}	2.85×10^{4}	2.85×10^{4}
A_2 (m^2)	**K_{c21} (kN/m)**	**K_{c22} (kN/m)**	**K_{c2} (kN/m)**	**K_{c2x} (kN/m)**	**K_{c2y} (kN/m)**
6.84×10^{-4}	1.15×10^{5}	6.20×10^{4}	4.02×10^{4}	2.85×10^{4}	2.85×10^{4}

Table 3.1 Stiffness K_e and the stiffness K_{sp} provided by the nail. (Ciocci, 2015).

K_e (kN/m)	**K_{ex} (kN/m)**	**K_{ey} (kN/m)**	**K_{sp} (kN/m)**	**K_{spx} (kN/m)**	**K_{spy} (kN/m)**
8.48×10^{4}	6×10^{4}	6×10^{4}	1.34×10^{3}	944	944

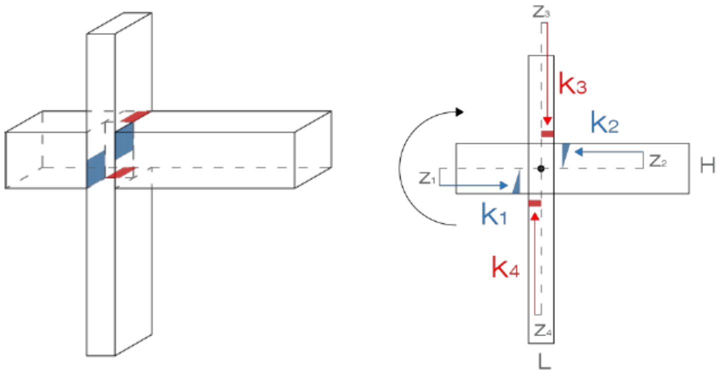


Figure 3.1 (a) Contact areas of the half-lap connection. (b) Rotational stiffness of the connection. (Ciocci, 2015).

Table 3.1 Rotational stiffness for the half–lap connection. (Ciocci, 2015)

A_1 (m^2)	**K_{11} (kN/m)**	**K_{12} (kN/m)**	**K_1 (kN/m)**	**Z_1 (m)**
9.60×10^{-3}	1.04×10^{6}	7.84×10^{4}	1.15×10^{4}	0.05
A_3 (m^2)	**K_{31} (kN/m)**	**K_{32} (kN/m)**	**K_3 (kN/m)**	**Z_3 (m)**
6.80×10^{-4}	5.18×10^{5}	4.24×10^{4}	3.92×10^{4}	0.02

To develop the numerical models representing the different MODs, link elements with an assumed zero length were used to model the semi-rigid connections. Each link element is assumed to be composed of six separate springs, one for each deformational degree of freedom (u1, u2, r3) (Ciocci, 2015). Two link elements were considered. Link 1 is used to represent the connection between the diagonal elements and the main frame. While, Link 2 is used to represent the connection between beams and posts. For Link 1 the axial and shear stiffness are specified, while for Link 2 the rotational stiffness is defined. In MOD 2 a linear elastic force-deformation relationship is used for Link 1, as shown in Figure 3.5. While, in MOD 3 a linear and a nonlinear elastic force-deformation relationship is used for Link 1 and Link 2, as shown in Figure 3.6. For link 1, after reaching the maximum capacity, the load progressively decreased to a null value until the ultimate displacement (Poletti et al.,2016). The maximum capacity of the connection was determined by the analytical procedure by Ciocci (2015), previously explained, and the ultimate displacement was assumed to be the same of the half-lap connection. For Link 2, the results obtained for the half-lap connection test by Poletti (2013) were considered to assume a numerical tri-linear behavior force-displacement diagram, shown in Figure 3.7, with the corresponding tri–linear moment–rotation diagram obtained. Table 3.6 shows a summary of the values for the stiffness of the diagrams, used to calibrate the model.

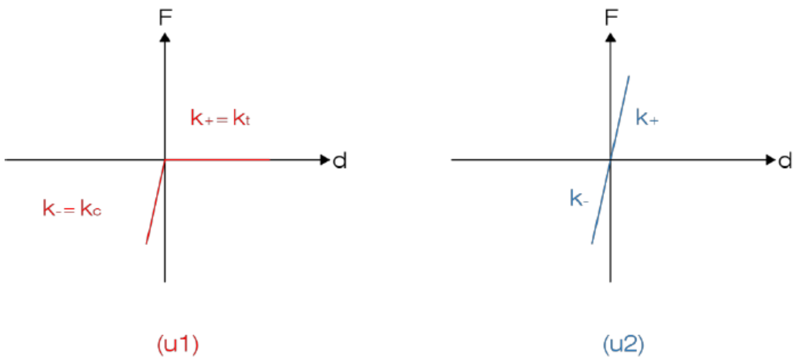


Figure 3.1 Linear elastic force–deformation for Link 1. (Ciocci, 2015).

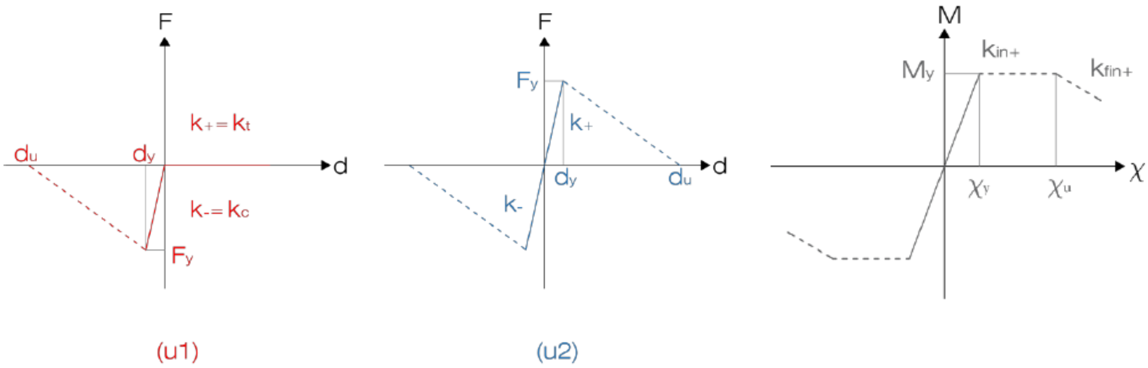


Figure 3.16 Linear and nonlinear elastic force–deformation: (a-b) Link 1. (c) Link 2. (Ciocci, 2015).

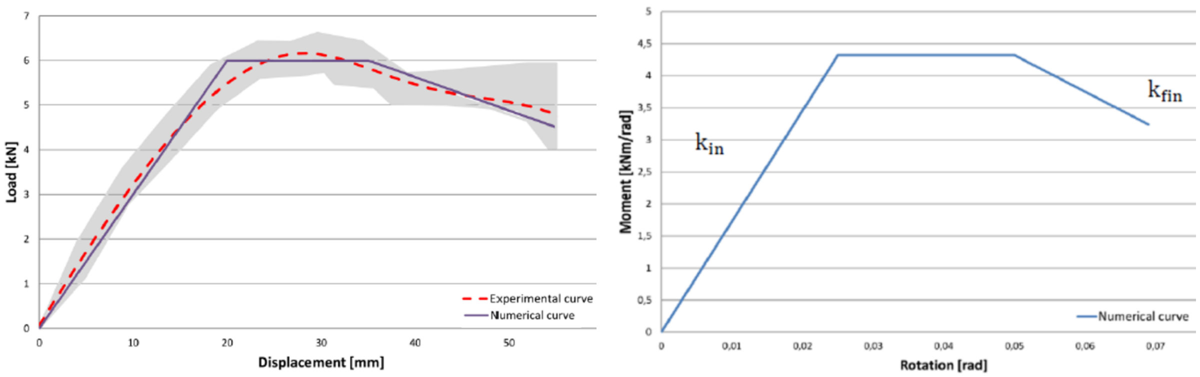


Figure 3.1 Half-lap connection: (a) Force–displacement diagram. (b) Moment–rotation diagram. (Ciocci, 2015).

Table 3.1 Values for the spring stiffness diagrams.

Link 1	u1	k1+ [Kn/m]	0
		k1- [Kn/m]	1.08E+04
		Fy [kN]	31
	u2	k2+ = k2- [kN]	1.08E+04
		Fy [kN]	31
	r3	k3+ = k3-[kNm/rad]	0
LINK 2	u1	k1+=k1- [Kn/m]	∞
	u2	k2+=k2- [Kn/m]	∞
	r3	k in3+[kNm/rad]	171
		k fin3+[kNm/rad]	47

The two-dimensional numerical model calibrated by SAP2000 considered linear elastic beam elements to model the timber frame and concentrated hinges to model the nonlinear behavior of the carpentry connections, as shown in Figure 3.8(a). The analysis consisted of a nonlinear static (pushover) analysis that was performed by incrementing the lateral load applied to the frame, causing structural elements to yield and the structure to experience a loss in stiffness. The obtained capacity of the frame is represented by the force-displacement curve shown in Figure 3.9. The two-dimensional model calibrated by DIANA FEA considered the same assumptions, except that the nonlinear behavior of the carpentry connections was modeled by spring elements, as shown in Figure 3.8(b). The obtained capacity of the frame is represented by the force-displacement curve shown in Figure 3.9. The initial stiffness of the frame calibrated by SAP2000, resulted in 2,60E+03 kN/m. While, for the model calibrated by DIANA FEA, resulted in 2,61E+03 kN/m. The models resulted with a good initial stiffness approximation when compared with the initial stiffness of the experiment calculated by Poletti (2013), which resulted in 2,60E+03 kN/m, by taking into account the origin and the point corresponding to 40% of the maximum load. However, the maximum resistance of the frame resulted to be different in both cases, as seen in Figure 3.9. This discrepancy may be due to differences in the computational process of the softwares, since the input data utilized for both models was the same.

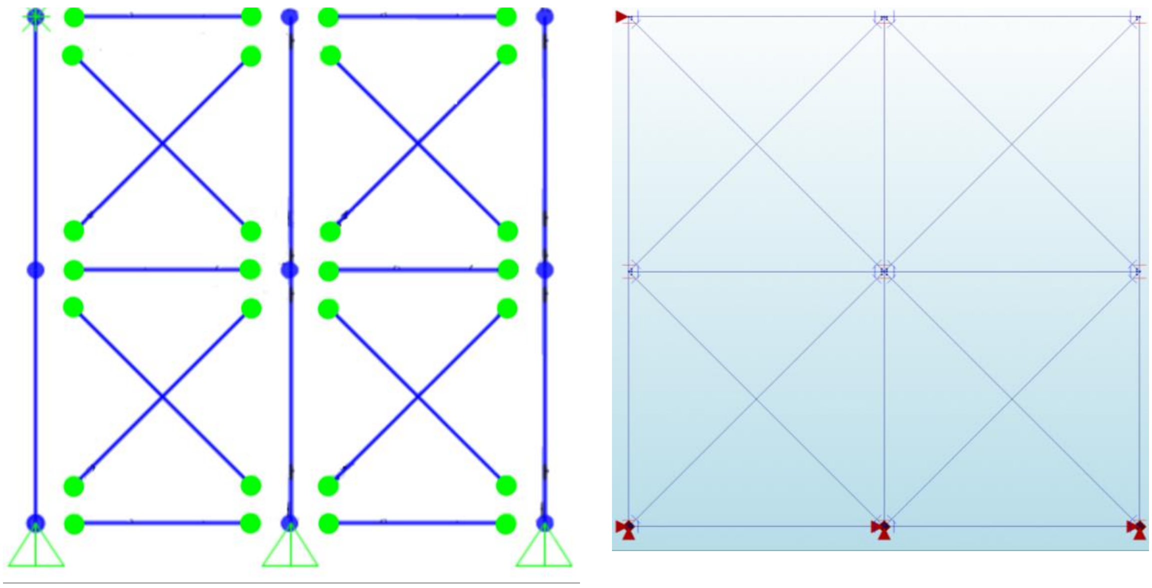


Figure 3.1 Modelling of nonlinear connections: (a) Concentrated hinges in SAP2000. (b) Spring elements in DIANA FEA.

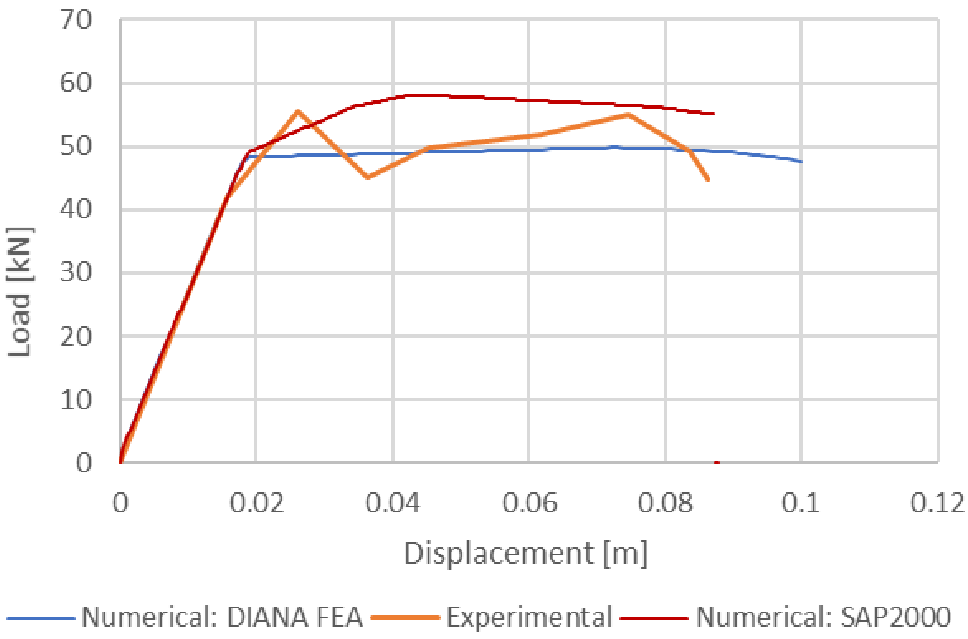


Figure 3.1 Numerical load-displacement capacity curves developed by SAP2000 and DIANA FEA, compared with the experimental results of the Pombalino frame tested by Poletti (2013).

To complete and validate the calibration of the frame, the collapse mechanism was studied and compared for both softwares. The points were the curve shows a change in slope are studied in detail, since they will characterize the global behavior of the frame. For the model analyzed in SAP2000 the points (A-E) shown in Figure 3.10 were studied. The legend shown in Figure 3.11 characterizes the behavior of each hinge in order to understand when each is formed in the frame throughout the analysis. During the analysis, the translational shear hinges do not reach a nonlinear behavior. Due to this, only the behavior of the translational axial hinges is shown in the collapse mechanism diagrams. Point A occurs when nonlinear behavior starts to take place in the frame. At this point most of the hinges of the diagonals reach the yielding resistance along the axial direction, as shown in Figure 3.12(a). At point B the hinges of the connection between the central beam and the outer posts reaches yielding under flexure, as shown in Figure 3.12(b). At point C most of the hinges of the connections between the beams and posts reach flexural yielding, as shown in Figure 3.12(c). Finally, at point D the hinges of the connection between the low beam and the internal post reach flexural yielding, as shown in Figure 3.12(d). The final collapse mechanism is shown in Figure 3.12(e).

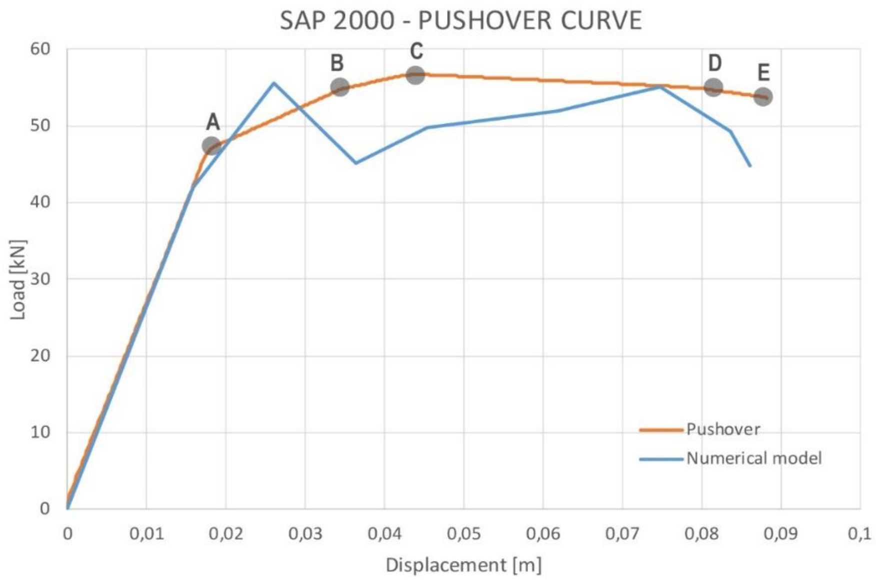


Figure 3.1 Pushover curve of Pombalino modeled by SAP2000.

a) b) c)

Figure 3.1 Hinges plastic deformation.

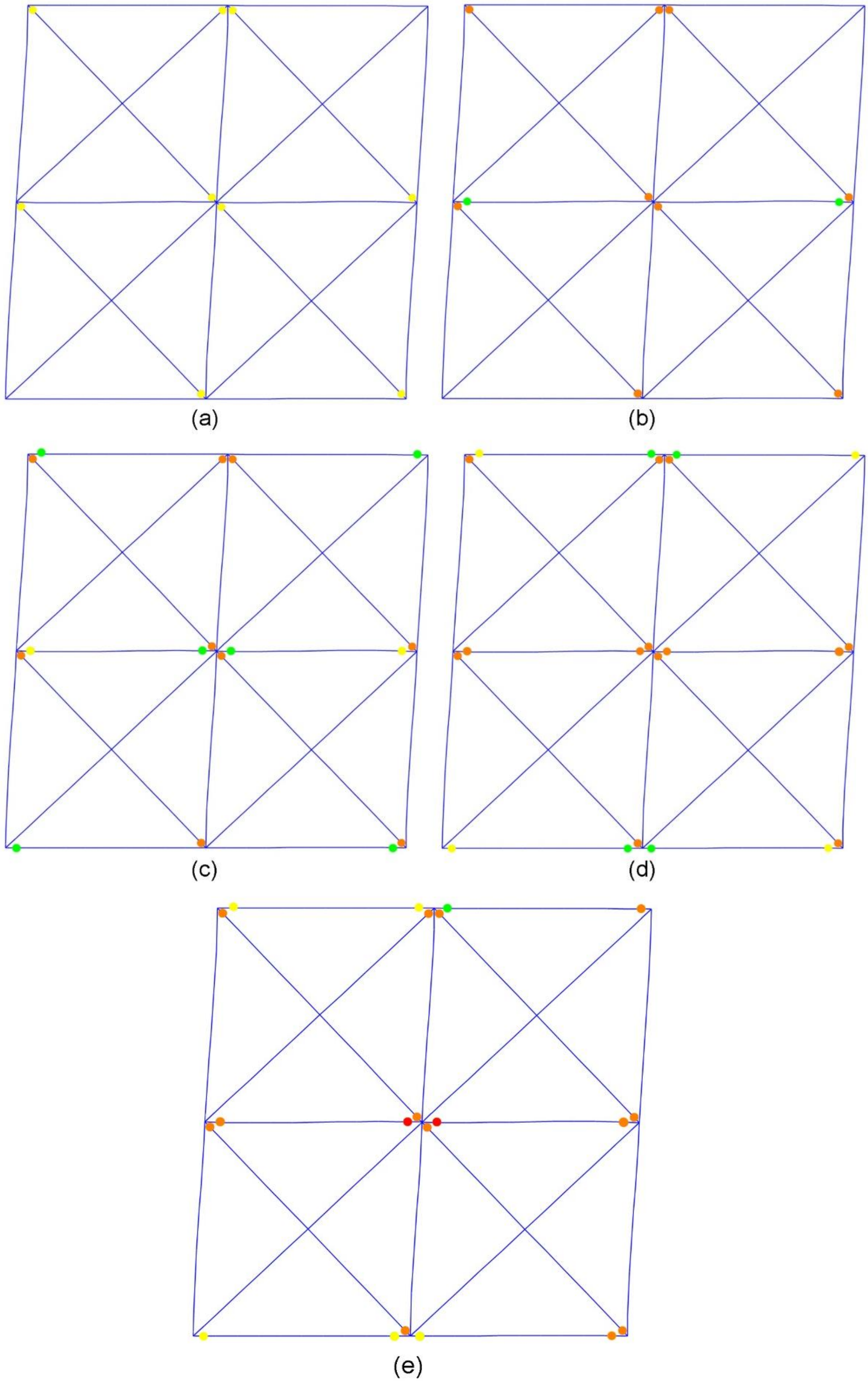


Figure 3.1 Hinges formation in each characteristic point of the Pombalino model by SAP2000.

For the model analyzed in DIANA FEA the points (A-E) shown in Figure 3.13 were studied. The legend previously shown in Figure 3.11, characterizes the behavior of each hinge in order to understand when each is formed in the frame throughout the analysis. In this case also, the translational shear hinges do not reach a nonlinear behavior. Due to this, only the behavior of the translational axial hinges is shown in the collapse mechanism diagrams. Point A occurs when nonlinear behavior starts to take place in the frame. At this point the hinges of the diagonals reach the yielding resistance along the axial direction, as shown in Figure 3.14(a). At point B occurs the same situation, as shown in Figure 3.14(b). At point C and point D the connections between the beams and posts reach flexural yielding, as shown in Figure 3.14(c) and (d). The final collapse mechanism is shown in Figure 3.14(e).

Comparing the collapse mechanism of the numerical models with the description of the experiment is concluded that both correspond to the experimental observations by Poletti (2013). An early failure of the central connections occured. The right connection failed, due to shear action of the diagonals and to stresses redistribution. Separation between the diagonals and the main frame occured by cracking at the central connection. And at the end of the tests, permanent deformation occured for all diagonals.

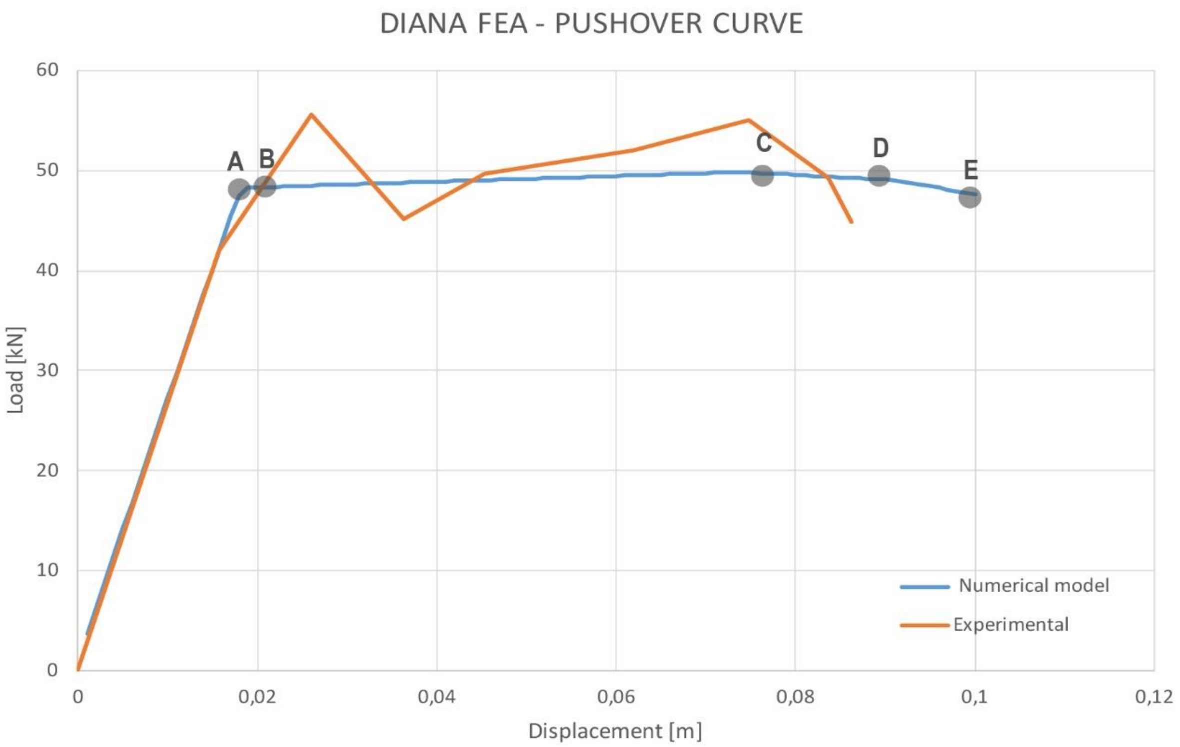


Figure 3.1 Pushover curve of Pombalino model by DIANA FEA.

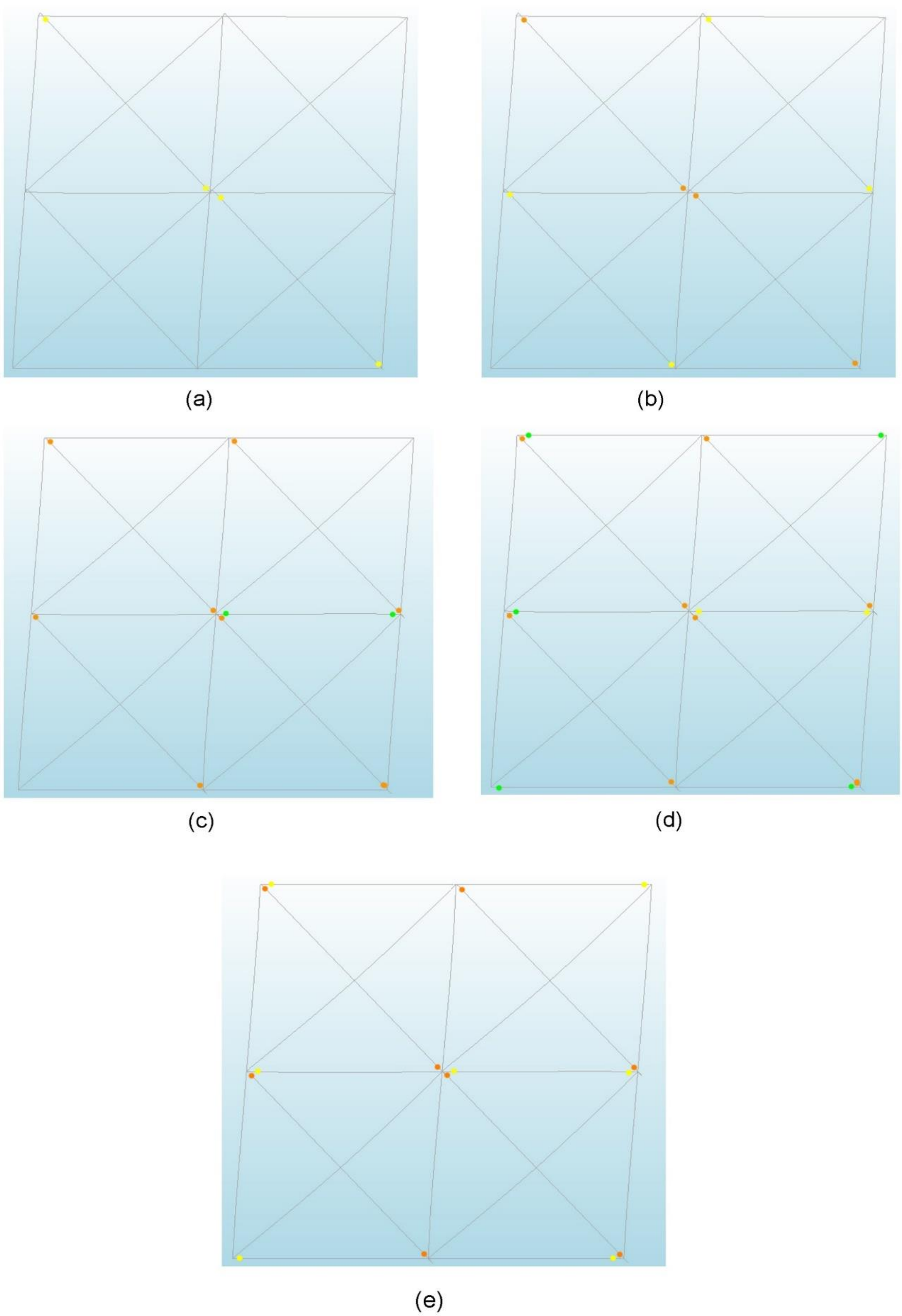


Figure 3.1 Hinges formation in each characteristic point of the Pombalino model by DIANA FEA.

3.2 Quincha Timber Frame Calibration

This second calibration consists in simulating the mechanical behavior of the Quincha timber frame reported in the experimental work by Quinn (2017). The timber frame wall is modeled by using the finite element analysis softwares DIANA FEA and SAP2000. To validate the accuracy of the softwares and the calibration, the numerical capacity curves obtained by both models are compared with the experimental capacity curve. Finally, the collapse mechanism of the frame is studied and compared for both softwares.

The two-dimensional model considers the frame geometry, material properties, loading and boundary conditions. The geometry of the timber frame is composed of the elementary cell shown in Figure 3.15(a), with the dimensions described in Table 3.7. The material of the frame is Cypress, except for the low beam that is Sapelli. Both materials are modeled as linear elastic isotropic homogeneous with the properties described in Table 3.8. The loading conditions include a distributed load of 3.79 kN/m applied downwards at the top beam and a horizontal displacement of 0.1 m applied at the left corner of the top beam. The boundary conditions include a restriction in horizontal and vertical directions at the nodes of the base. Also, a restriction in the horizontal direction is added at the node where the prescribed deformation is applied, as shownin Figure 3.15(b). For both modelling softwares, linear elastic beam elements are considered to represent the timber frame. However, the nonlinear behavior and finite stiffness of the carpentry joints is modeled by applying springs elements in DIANA FEA and concentrated hinges in SAP2000. The carpentry joints of the Quincha timber frame include: mortise and tenon joints and lap joints, as described in Section 2.1.

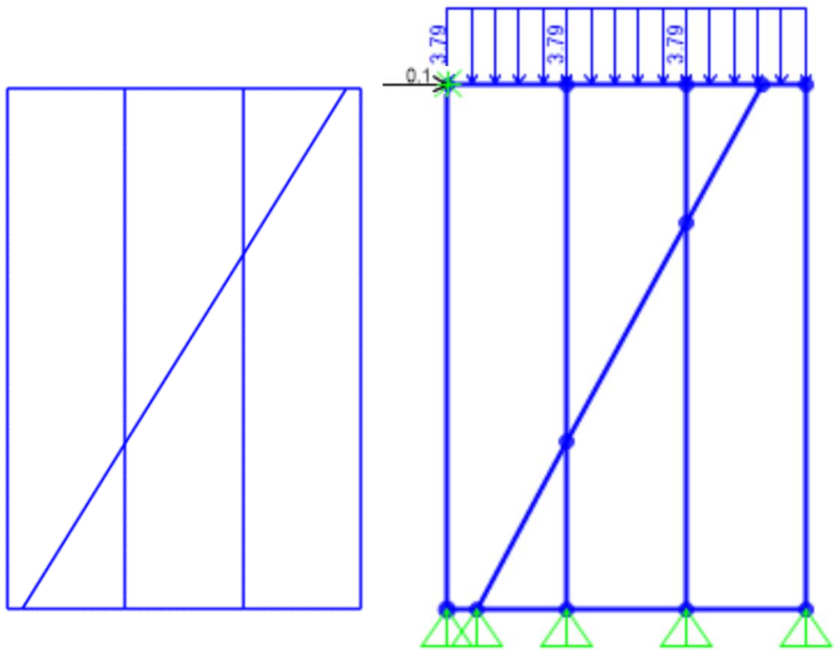


Figure 3.2 Elementary cell of the Quincha timber frame. (a) Geometry. (b) Loading and boundary conditions

Table 3.2 Cross sectional dimensions of the elements of the frame. (Quinn, 2017)

Elements	Height (mm)	Width (mm)
Beams	60	80
Posts	60	80
Diagonal	90	30

Table 3.2 Material Characteristics of Sapelli and Cypress. (Quinn, 2017)

Wood	Sapelli	Cypress
E (kN/m^2)	7.7E+06	6.4E+06
ρ (kg/m^3)	400	390
ν	0.3	0.3

The first step of the calibration process is to analyze a rigid model and then create other models by adding the nonlinear behavior of the connections. The models were analyzed by:

- MOD 0 – Rigid connections
- MOD 1 – Hinged connections
- MOD 2 – Semi-rigid connections: rotational stiffness of the mortise and tenon
- MOD 3 – Semi-rigid connections: rotational and translational stiffness of the mortise and tenon
- MOD 4 – Semi-rigid connections: rotational and translational stiffness of the mortise and tenon, and translation of the lap joint.

The first model analyzed is MOD 0 which considers all the connections as perfectly rigid. The second, MOD 1, considers hinged connections by releasing all the rotational DOFs at one end. The next models, MOD 2, MOD 3, and MOD 4 consider the connections as semi-rigid by introducing a spring stiffness value at the DOFs.

The behaviors of the semi-rigid connections are introduced in the model by considering force-deformation diagrams. The behavior of the mortise and tenon connections between the beams and the posts are represented with axial and rotational stiffness. While, the behavior of the lap connections between the diagonal and the beams of the main frame are represented with axial stiffness. The axial stiffness of the mortise and tenon connection, shown in Figure 3.16, was based in the experimental data of vertical uplift measured for the frame by Quinn (2017). The rotational stiffness of the mortise and tenon connection, shown in Figure 3.17, was based in the results obtained from the separate specimen of the connection that was tested by Quinn (2017). For the lap connection the stiffness provided by the nail was calculated based on the Eurocode equation for nails without pre–drilling, obtaining the behavior shown in Figure 3.18, with a maximum capacity of 2.15 kN.

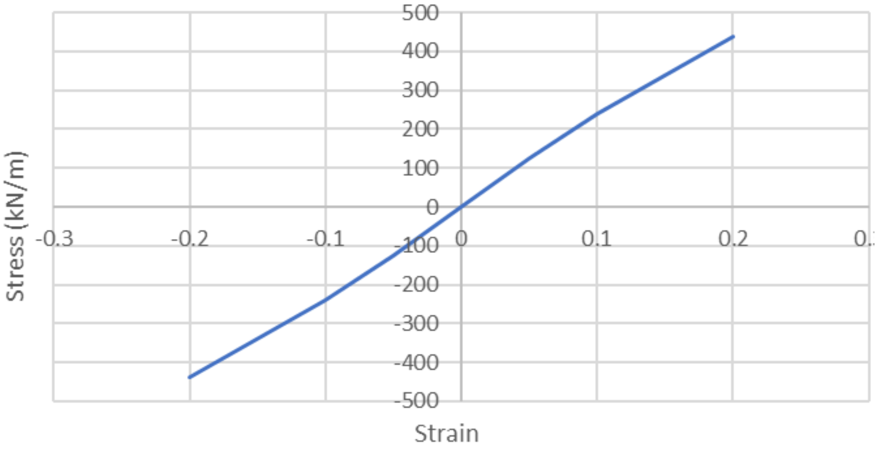


Figure 3.2 Mortice and tenon connection translational stiffness. (Quinn, 2017)

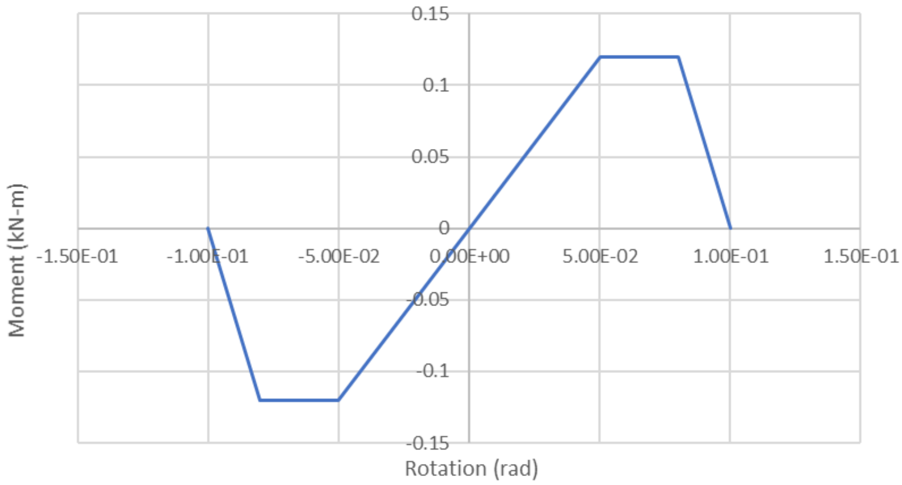


Figure 3.2 Mortise and tenon connection rotational stiffness. (Quinn, 2017)

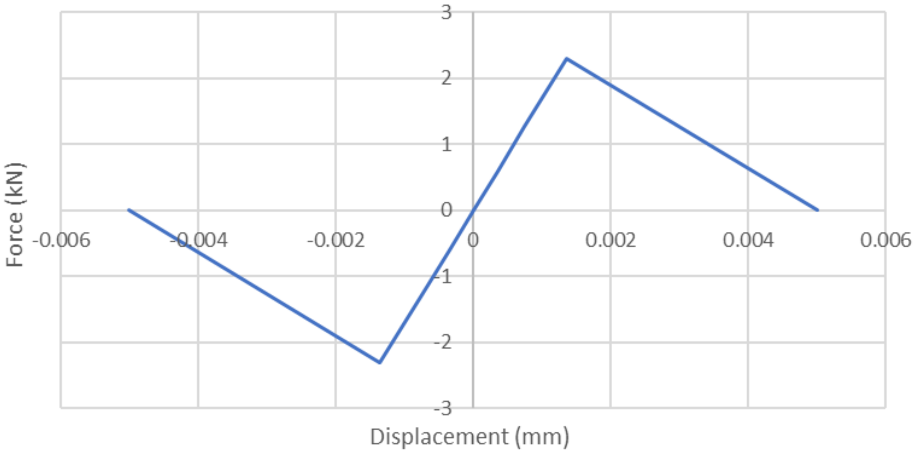


Figure 3.2 Lap connection capacity. (Quinn, 2017)

The two-dimensional numerical model calibrated by SAP2000 considered linear elastic beam elements to model the timber frame and concentrated hinges to model the nonlinear behavior of the carpentry connections, as shown in Figure 3.19(a). The analysis consisted of a nonlinear static (pushover) analysis. The obtained capacity of the frame is represented by the force-displacement curve shown in Figure 3.20. The two-dimensional model calibrated by DIANA FEA considered the same assumptions, except that the nonlinear behavior of the carpentry connections was modeled by spring elements, as shown in Figure 3.19(b). The obtained capacity of the frame is represented by the force-displacement curve shown in Figure 3.20. The initial stiffness of the frame calibrated by SAP2000, resulted in 178 kN/m. While, for the model calibrated by DIANA FEA, resulted in 179 kN/m. The models resulted with a good initial stiffness approximation when compared with the initial stiffness of the experiment calculated by Quinn (2017), which resulted in 180 kN/m. However, similar to the previous case of modelling calibration, the maximum resistance of the frame resulted to be different, as seen in Figure 3.20. This discrepancy may be due to differences in the computational process of the softwares, since the input data utilized for both models was the same.

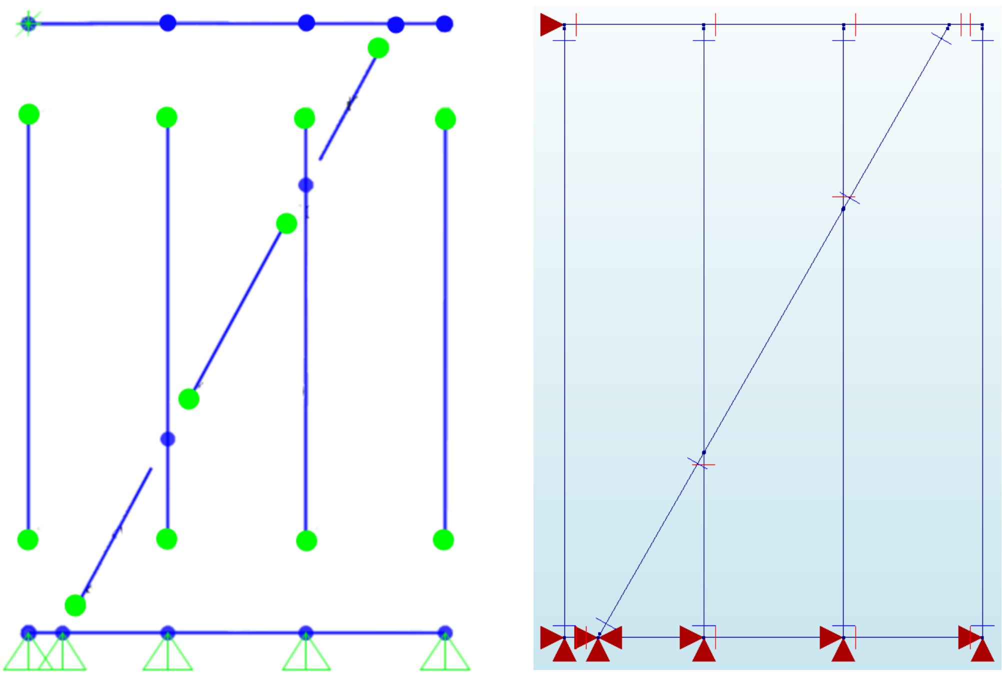


Figure 3.2 Modelling of nonlinear connections: (a) Concentrated hinges in SAP2000. (b) Spring elements in DIANA FEA.

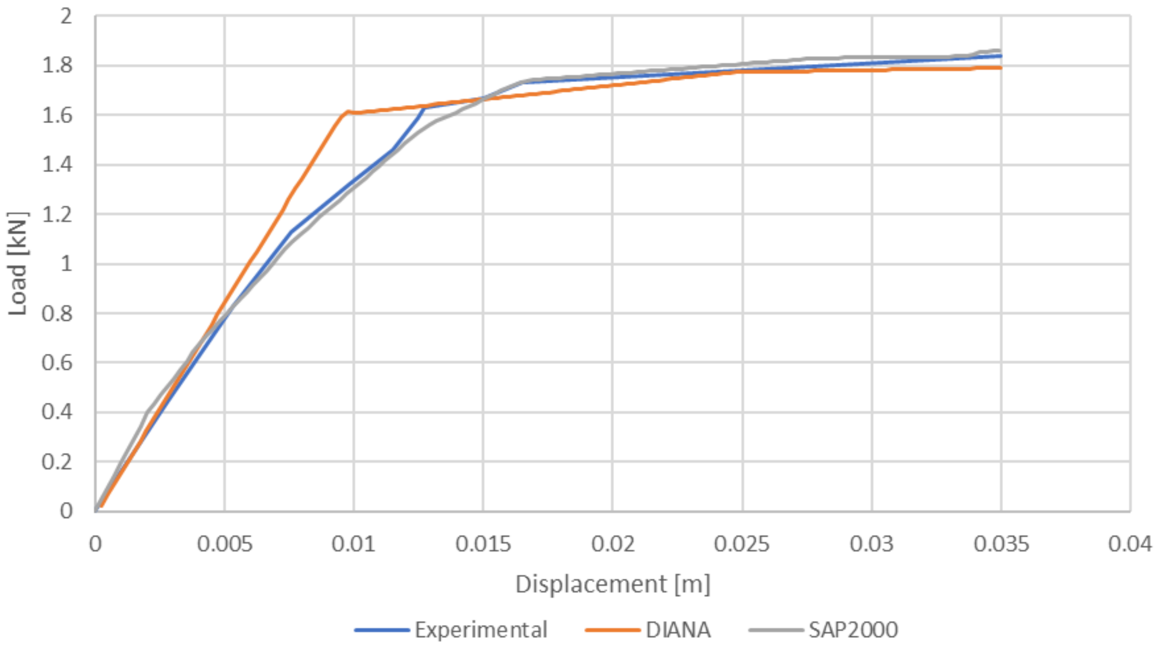


Figure 3.2 Numerical load-displacement capacity curves developed by SAP2000 and DIANA FEA, compared with the experimental results of the Quincha frame tested by Quinn (2017).

To complete and validate the calibration of the frame, the collapse mechanism was studied and compared for both softwares. The points were the curve shows a change in slope are studied in detail, since they will characterize the global behavior of the frame. For the model analyzed in SAP2000 the points (A-D) shown in Figure 3.21 were studied. The legend shown in Figure 3.22 characterizes the behavior of each hinge in order to understand when each is formed in the frame throughout the analysis. Point A occurs when nonlinear behavior starts to take place in the frame. At this point the hinges between the diagonal and the internal posts reach the yielding resistance along the flexural direction, as shown in Figure 3.23(a). At point B three of the hinges between the top beam and the posts reach yielding, as shown in Figure 3.23(b). At point C all the hinges between the top beam and the posts reach yielding, as shown in Figure 3.23(c). The rotational hinges that are generated between the top beam and the posts do not reach collapse. The final collapse mechanism is shown in Figure 3.23(d).

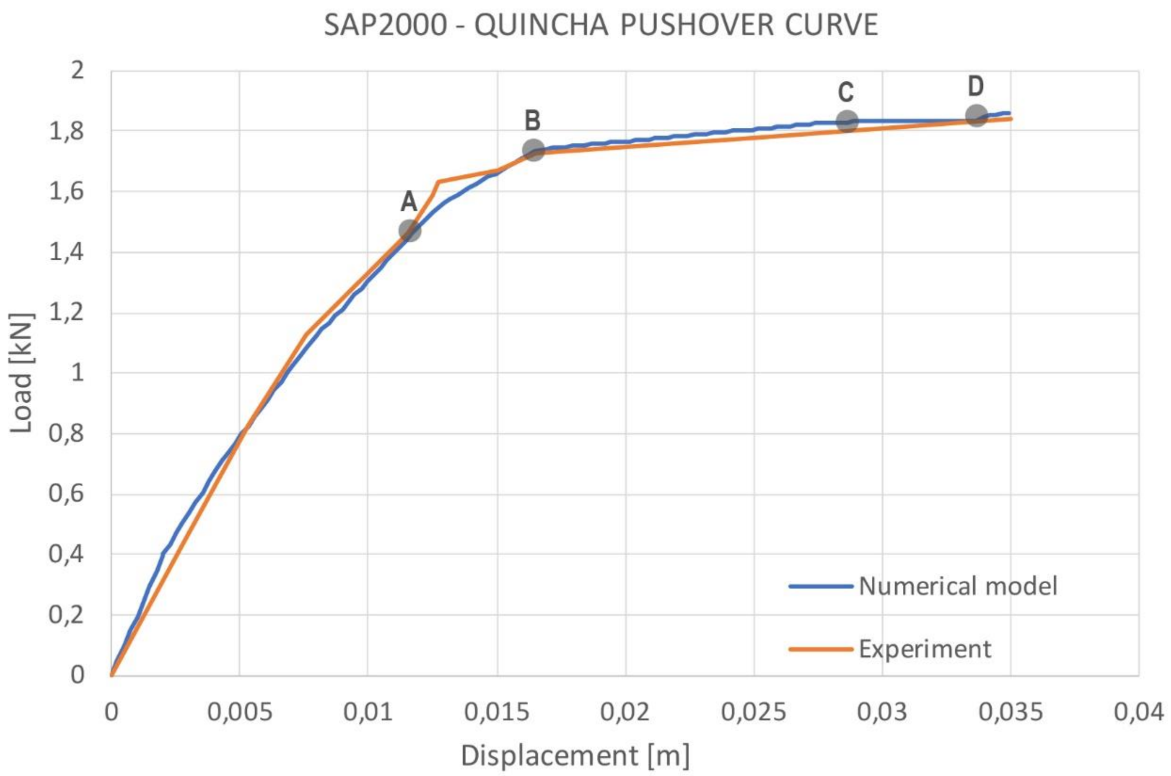


Figure 3.2 Pushover curve of Quincha model by SAP2000 with characteristic points of the global response.

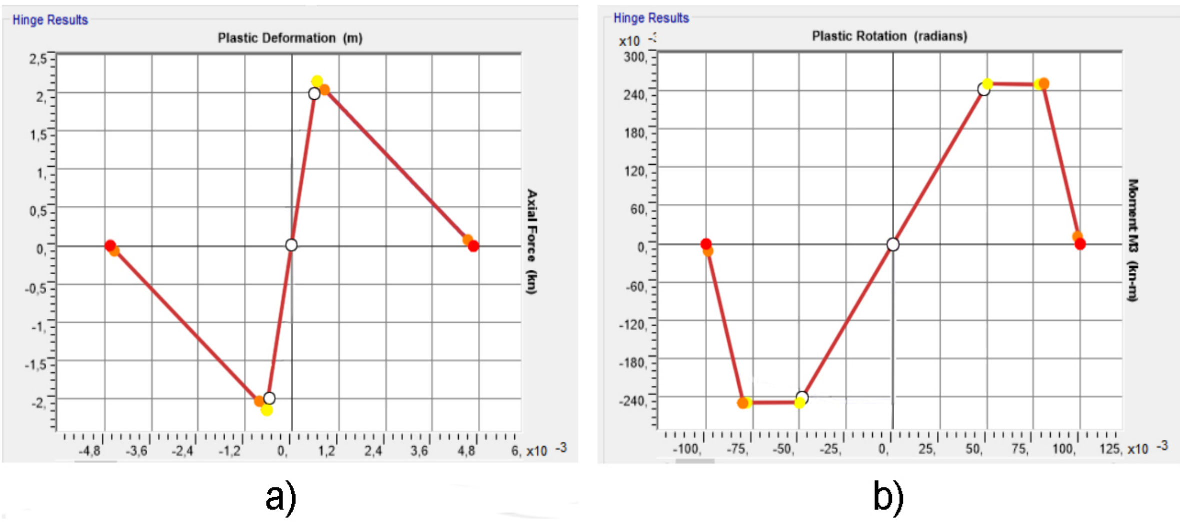


Figure 3.2 Hinges plastic deformation.

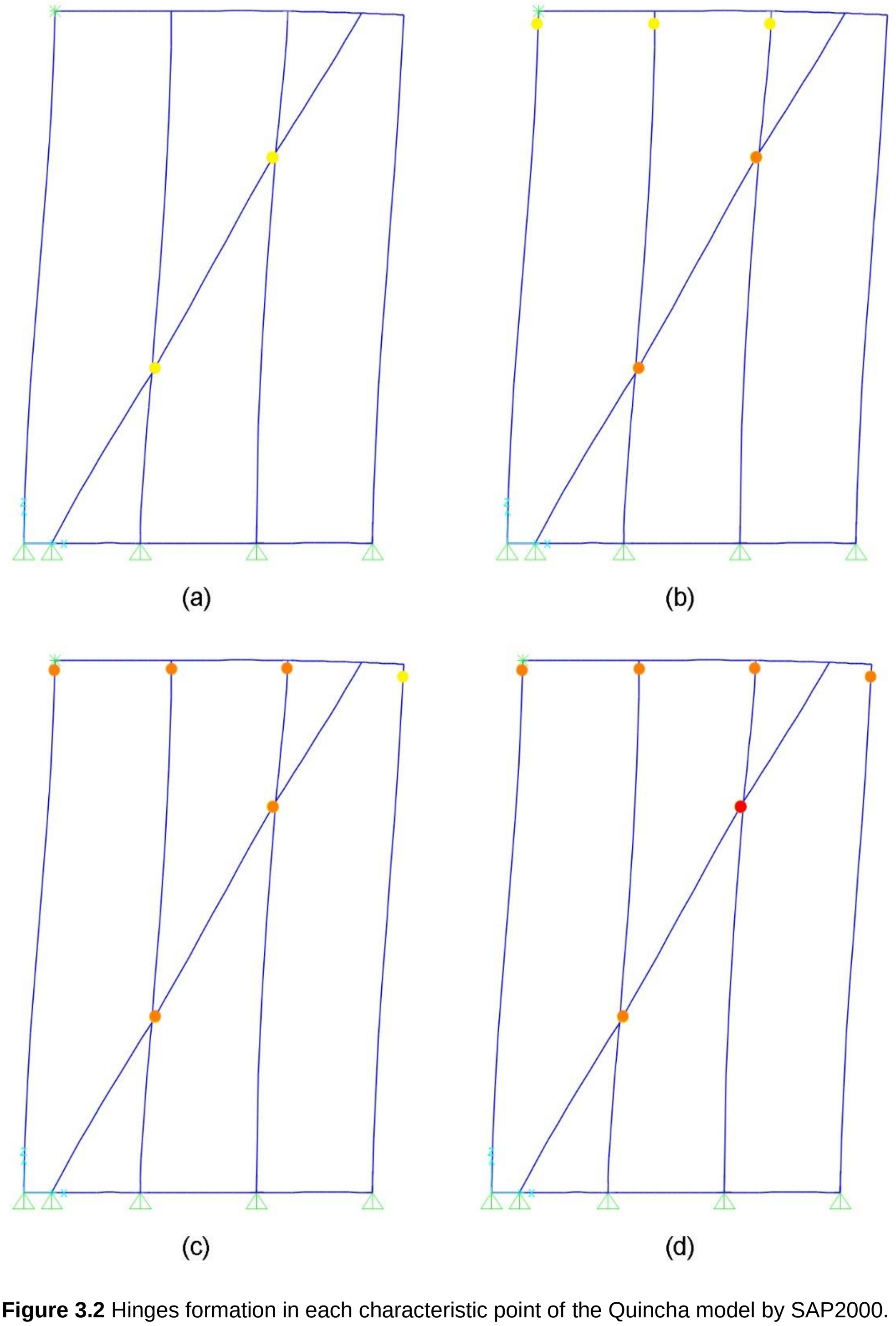


Figure 3.2 Hinges formation in each characteristic point of the Quincha model by SAP2000.

For the model analyzed in DIANA FEA the points (A-D) shown in Figure 3.24 were studied. The legend previously shown in Figure 3.22, characterizes the behavior of each hinge in order to understand when each is formed in the frame throughout the analysis. Point A occurs when nonlinear behavior starts to take place in the frame. At this point the hinges between the diagonal and the internal post, and between the diagonal and the top beam reach the yielding resistance along the flexural direction, as shown in Figure 3.25(a). At point B all the hinges of the connections of the diagonal reach yielding, as shown in Figure 3.25(b). The final collapse mechanism is shown in Figure 3.25(c).

Comparing the collapse mechanism of the numerical models with the description of the experiment is concluded that the models correspond with the experimental observations by Quinn (2017). First, yielding of the connection between the diagonal and frame occur. The diagonal pulls downwards on the top plate, splitting the beam perpendicular to the grain and then gradual failure of top plate perpendicular to the grain occur. Visible inplane bending of diagonal was observed, as well as a brittel failure of diagonal in tension.

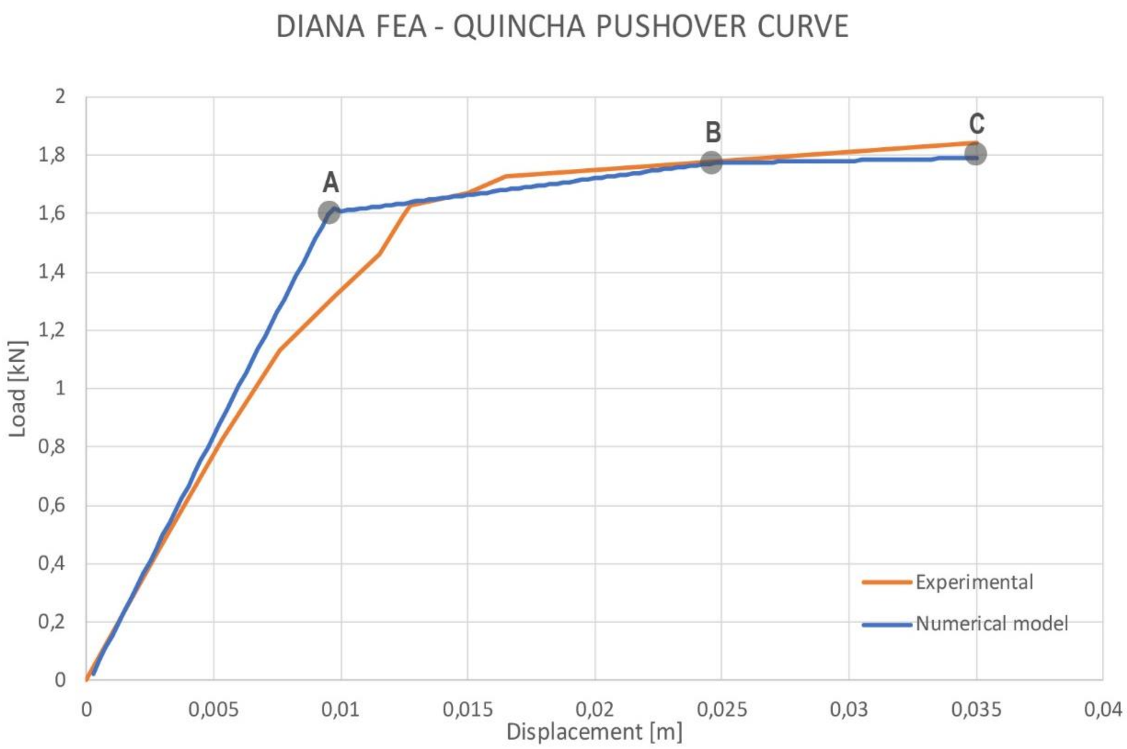


Figure 3.2 Pushover curve of Quincha model by DIANA FEA with characteristic points of the global response.

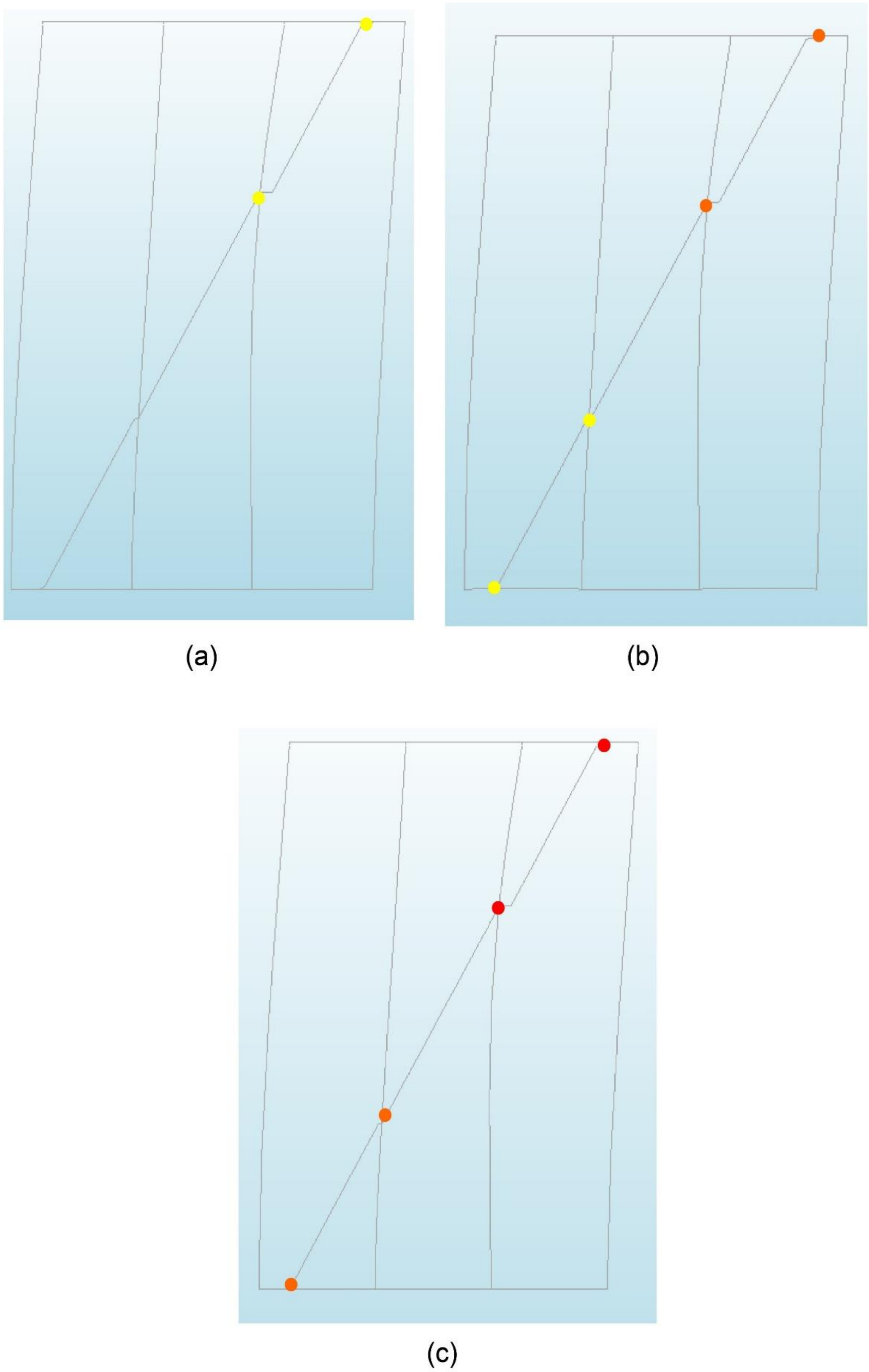


Figure 3.2 Hinges formation in each characteristic point of the Quincha model by DIANA FEA.

CHAPTER 4 HISTORICAL TIMBER FRAME HOUSES OF VALPARAISO

4.1 Historical and Context Review

Timber frame constructions were introduced in the city of Valparaíso between the late nineteenth and early twentieth centuries. In this period, the port of Valparaíso was one of the most important of the Pacific Ocean. The large growth of the port was reflected in the transformation of the city, which experienced many changes in social and cultural life. The flourishing economy of the city attracted numerous immigrants to populate the area. British, Italian, German, and North American communities were the most important colonies in this period (Tornedo, 1872). Most of them were affluent families, which brought new lifestyle concepts, as well as architectonic stylistic resources and advanced building technologies. The influence of the foreign population in the city was reflected in the change of the urban and architectonic environment. Mostly British, German and Italian communities established their houses in the *Cerros Alegre* and *Concepción* (Figure 4.1). These neighborhoods were marked by a European environment, configured of Victorian houses of great architectonic value, front yards, and cobbled streets. Most of these houses were made of timber frames and masonry, combining foreign technologies and local construction techniques. Timber-frame techniques were widely spread through all the territory of Valparaíso and became one of the most representative structural system used for housing. However, the pioneer’s timber-frames are highly influenced by foreign models. Prefabricated timber houses were also brought from Europe and sold in the newspaper (Jiménez, 2015).

Figure 4.1. Photography by Felix Le Blanc. (a) Cerro Alegre and (b) Cerro Concepción at the end of nineteenth century (date unknown) (Jiménez, 2015)

Valparaiso, recognized by UNESCO as a World Heritage (Figures 4.2 and 4.3), represents an exceptional case-study due to the vernacular condition of it timber frame constructions. This is reflected in a great variability of structural systems layouts which are adapted to the local requirement of the city. For example, in the hills *Cerros Alegre* and *Concepción*, the vernacular architecture became innovative by adapting the structure to a complex geographic condition. Also, the high seismicity of the territory represents a large concern for the buildings of Valparaíso due to their potential structural damages and consequences in the urban areas.

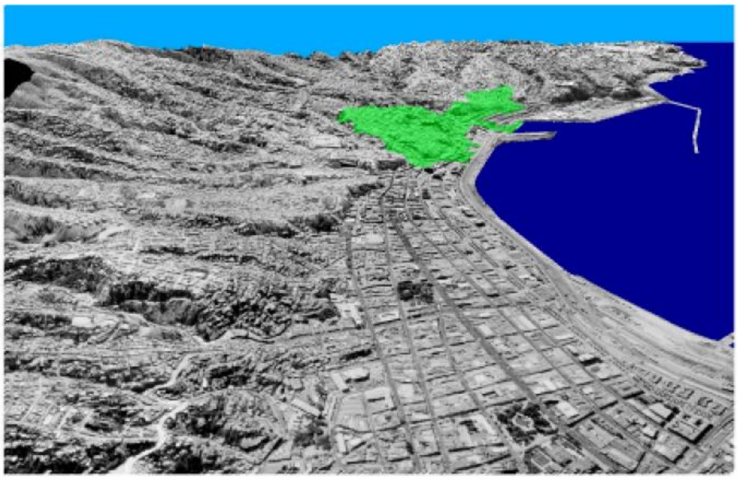


Figure 4.1 Three-dimensional view of Valparaiso historical zone marked in green. Author: Pedro Muñoz Aguayo – Geographer MINVU (2002). (UNESCO, 2002)

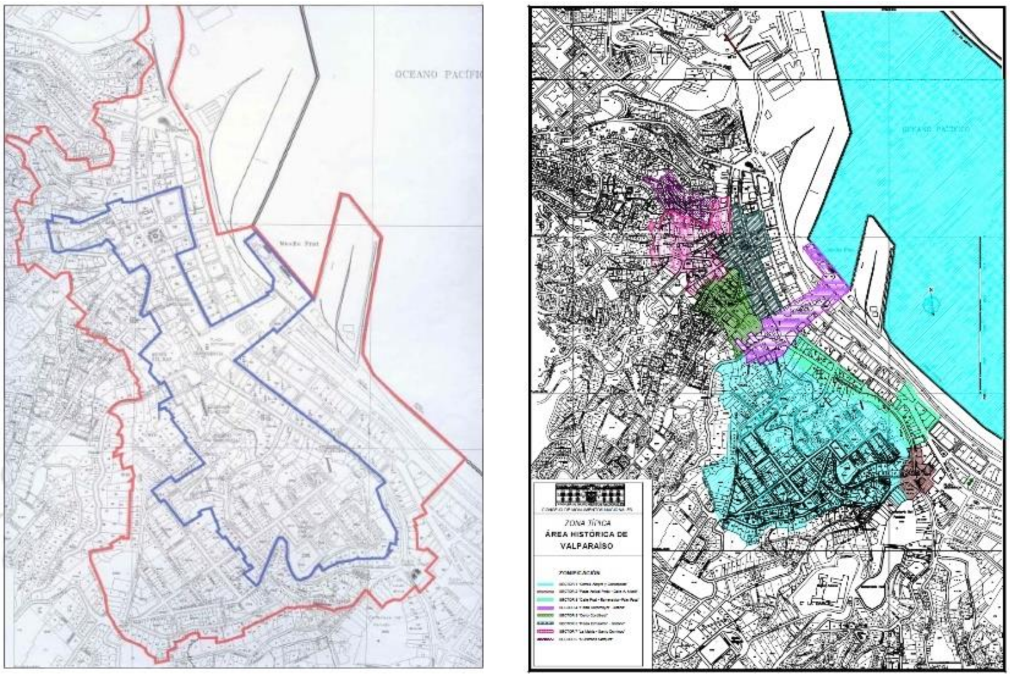


Figure 4.1 (a) Historical zone of Valparaiso marked by red and the world heritage zone marked by blue. (b) Zone division of the Historical area of Valparaiso; Cerro Alegre and Cerro Concepcion marked by blue. (UNESCO, 2002)

4.2 Structural Configurations of Timber Frame Walls

The traditional timber frame structures in Valparaíso are mostly configured by platform frame systems. This system supposes the presence of platform stories between the upper and lower load-bearing walls. The stories are placed between the walls, and both are configured as independent frames. The structure of the story is configured by joists spaced at 0.4 m each, which are supported by the sills of the lower and upper walls through small notches (Figure 4.4).

The typical timber lateral load-bearing system in Valparaíso can be divided into two scales: an elementary cell and a shear wall. The elementary cell is composed by the minimum number of timber elements and carpentry joints that create a basic timber module. The shear wall is composed by different numbers of elementary cells according to the design of the wall. Finally, the whole structure is configured by shear walls and timber stories (Figure 4.5). In the following sections these scale divisions are used to characterize in detail the configuration of the typical load bearing-walls of Valparaíso.

Figure 4.2 (a-b) Platform frame system configuration. (c) Walls-story connection. (Jiménez, 2015)

Figure 4.2 (a) Elementary cell. (b) Shear wall. (c) Shear walls and timber stories. (Jiménez, 2015)

4.2.1 Elementary Cell

The elementary cell or modulus that configures a typical load-bearing walls is composed by two beams, three posts and one diagonal (Figure 4.6). Usually, the posts are spaced from 0.40 m to 0.60 m, having an average length of 3.6 m and a mortise and tenon carpentry joint connection to the beams. The diagonal element also has a length of 3.6 m and is located from the base corner up to 0.3 m before the end of the opposite post, with a nailed connection to the posts. The cross-section of these elements can vary from 0.10 m x 0.10 m to 0.15 m x 0.15 m, being 0.10 m x 0.15 m the most typical cross-section for façade elements. All the elements that compose an elementary cell or shear wall have the same cross-section.

Figure 4.2.1 Elementary Cell of the Valparaiso timber frame.

4.2.2 Shear Walls

A shear wall is a structural system, which purpose is to resist lateral forces that occur parallel to the wall direction. The shear or load-bearing walls of Valparaiso timber frames are composed of the elementary cells described previously. These elementary cells are combined according to the design of the wall. The shear wall configuration can vary including openings like doors and windows. These openings may vary in size but typically a door has a width of 1.2 m and a height of 3 m. However, a typical window opening has a width of 1.2 m and a height of 2 m.

A typical shear wall including a door opening is composed of two elementary cells, one at the left and one at the right of the door opening (Figure 4.7(a)). In the elementary cell at the left the diagonal element will work in tension towards resisting the lateral load; while in the elementary cell at the right the diagonal will work in compression. Similarly, the configuration of a typical shear wall including a window opening is composed of two elementary cells, one at the left and one at the right of the window opening (figure 4.7(b)). In this case the diagonal elements work in the same way as for of the door opening case.

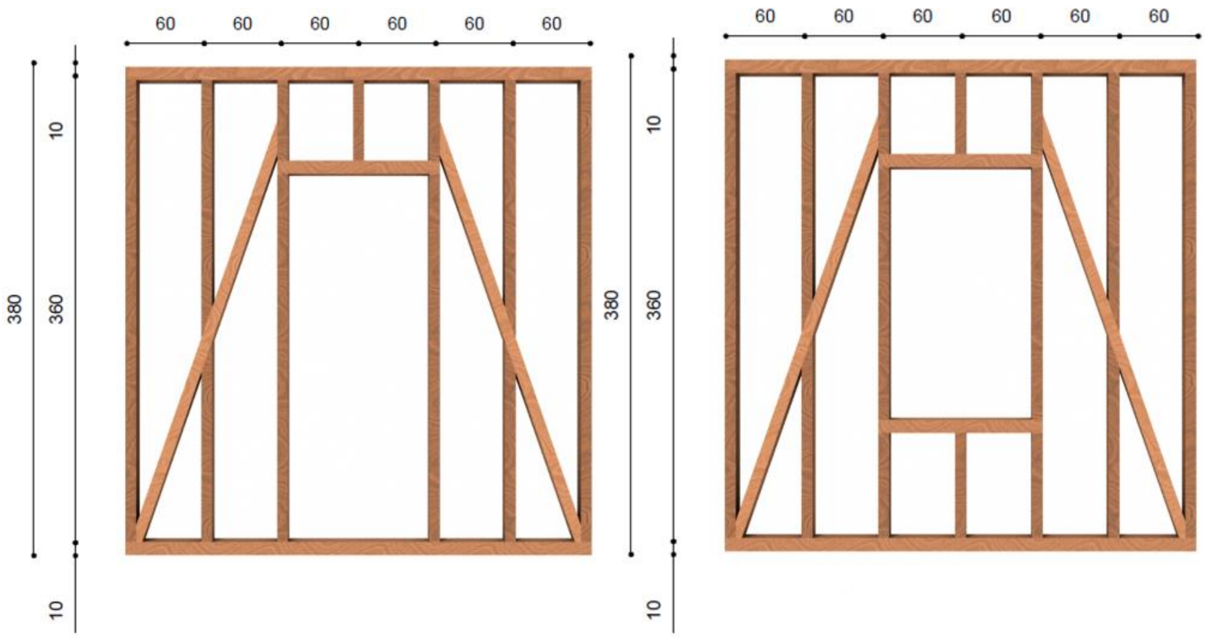


Figure 4.2.2 Shear walls: (a) Typical shear wall with a door opening. (b) Typical shear wall with a window opening. Dimensions in cm.

4.3 Traditional Carpentry Joints

Before the invention of nails or metal fasteners, timber structures constructions were characterized by the use of traditional carpentry joints. Valparaiso timber frames are characterized by these types of connections, which are a solution to joint timber elements without or with a reduced need of metal fasteners. If metal fasteners were added, their only function was to keep the elements together, but it did not provide any additional strength to the joint. These types of joints work by equilibrating forces transferring them from one element to another by friction or contact pressure. The timber frame from Valparaiso is characterized by three carpentry joints (Figure 4.8):

- **Mortise and tenon joints** connecting the posts and beams
- **Notched joint** connecting the diagonal with the central post
- **Connection by contact** connecting the diagonal with the external posts

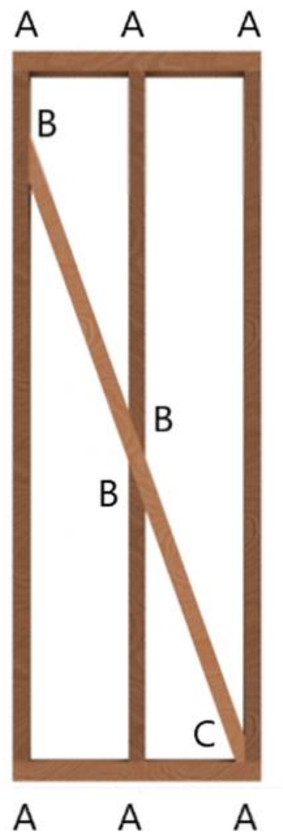


Figure 4.3 Joints location: A – Mortice and tenon joint, B – Nothced joint, C – Connection by contact.

4.3.1 Mortise and Tenon

Mortise and tenon carpentry joints are semirigid connections classified as ensemble joints. They are composed of two timber pieces, the mortise hole and the tenon tongue, with the purpose of connecting the vertical and transversal elements of the timber frame. These joints connect members that usually form an "L" or "T" type configuration (Branco, 2015). The two timber elements are connected by the tenon that is formed at the end of a member and is inserted into a square or rectangular hole cut, the mortise. They are designed to transfer compression and they depend of the contact surface. The tenon has the function of providing resistance to the joint by transferring the load through the surface that surrounds the tenon. To guarantee this behavior the tenon must have a shorter length than the depth of the mortise (Arriaga, 2011). This type of carpentry joint is one of the most common and its technology continued to expand causing variations in geometry. For the case of Valparaiso, the tenon had a length of 50 mm and a cross section of 95 mm x 38 mm, while the mortise had the same cross section of the tenon.

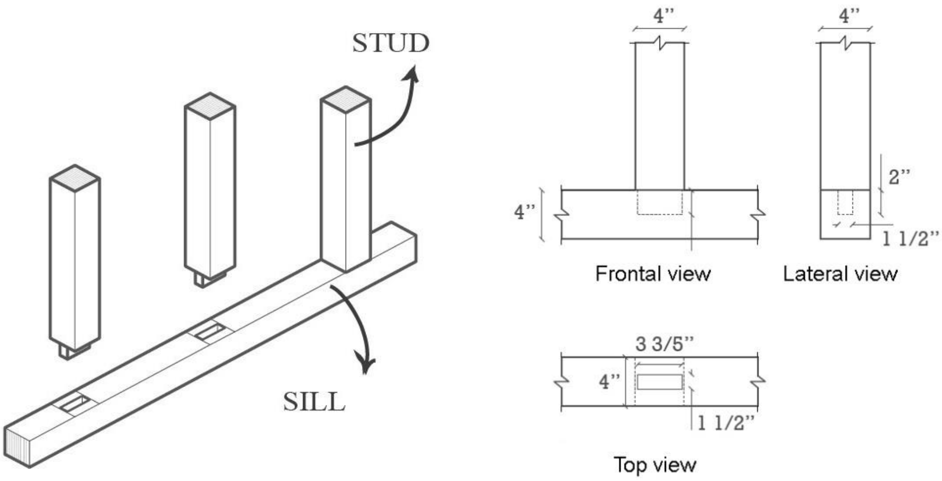


Figure 4.3.1 Mortise and tenon carpentry connection (a) Studs-sills connection. (b) Frontal and lateral view. (Jiménez, 2015)

Figure 4.3.1 Mortise and tenon photographs (a) Studs-sills connection. (b) Girt-stud connection. (Jiménez, 2015)

4.3.2 Notched Connection

The notched connection is a simple and common one. This type of joint is linked to the development of king post and similar frames (Branco, 2015). It consists of joining two pieces, one piece compressed in another through a notch (Arriaga, 2011). A notch is a "V" shape groove generally perpendicular to the length of the element that is going to be connected to (Figure 4.11). In Valparaiso timber frames this connection is used for joining the diagonal element with the internal post. In this case the notch is located in the post and is what creates the strength of the joint by transferring the load to the diagonal or central element. This connection is also used for joining the diagonal upper end with the external post. In this case the notch is located in the diagonal. For this type of connection, it is necessary to secure the pieces by nails, screws or ironworks.

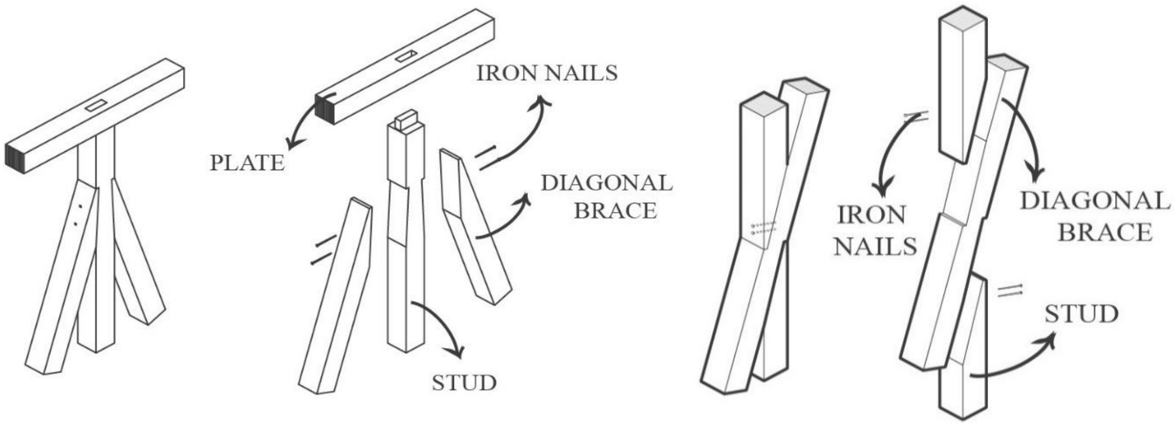


Figure 4.3.2 Notched carpentry joints (a) Upper diagonal-studs connection. (b) Central diagonal-stud connection. (Jiménez, 2015)

Figure 4.3.2 Notched connection photographs (a) Upper diagonal-studs connection. (b-c) Central diagonal-stud connection. (Jiménez, 2015)

4.3.3 Connection by Contact

The connection by contact is a simple connection usually used to joint diagonal timber elements with vertical and horizontal elements. In the case of Valparaiso timber frames this joint is used to connect the lower end of the diagonal element with the post and lower beam (Figure 4.13). The connection counts with two contact areas, one in contact with the post and another in contact with the lower beam. These contact areas are the ones that provides the strength to the joint. Another characteristic of this joint is that the connection requires the use of nails or metal fasteners to keep the joint together.

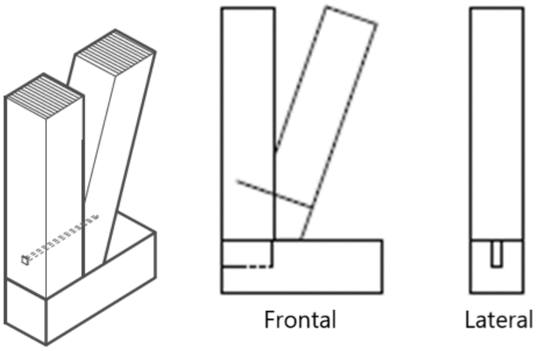


Figure 4.3.3 Connection by contact (a) Diagonal and timber frame connection. (b) Frontal and lateral view. (Jiménez, 2015)

Figure 4.3.3 Connection by contact photograph (Jiménez, 2015)

CHAPTER 5 NUMERICAL ANALYSIS OF VALPARAISO TIMBER FRAME SHEAR WALLS

5.1 Valparaiso Timber Frame

The following sections are focused into performing a numerical analysis of representative timber frame cases of the historical center of Valparaíso. The numerical analysis consisted of a nonlinear static analysis that considers the full load application of the dead and vertical loads, and a displacement control of the applied horizontal displacement. Other considerations including physical and mechanical properties were taken into account for the models. Several numerical analyses were performed:

- Elementary cell
- Window opening cell
- Door opening cell
- Shear walls – Case 1 and Case 2

5.1.1 Geometry and Materials

According to Jiménez (2015) the geometry of the timber frame from Valparaiso is composed of posts, beams and diagonal elements with sections that could vary from 0.10 m x 0.10 m to 0.15 m x 0.15 m. For the numerical models it was assumed a cross section of 0.10 m x 0.15 m, being this the most typical for façade elements. The configuration of the frame varies depending of the type and quantity of openings, and the number of floors, but the posts are spaced from 0.40 m to 0.60 m, having an average length of 3.6 m, as explained in Section 4.2.1.

The timber species most commonly used for traditional timber housing in Valparaíso in the studied period were the Chilean Oak and Oregon Pine, according to Jiménez (2015) and Jorquera (2016). However, these two materials have different mechanical characteristics. The Oak is denser than the Oregon Pine and has a higher elastic modulus as shown in Table 5.1. According to the structural inspections carried out by Jiménez (2015), Chilean Oak timber elements are commonly used to configure façade frames and joist stories, while Oregon Pine elements to configure the internal partitions. Nevertheless, and due to the vernacular conditions of timber frames houses in Valparaíso, the configuration of the building can vary. The assumption of either of these wood species as the constitutive material of timber frames in Valparaíso would lead to considerably different structural behaviors. Since the shear walls to study are façade frames, only Oak was considered for the numerical models.

Table 5.1.1 Material Properties of Chilean wood (Perez 1990 and INFOR 2010).

Wood Specie	Chilean Oak	Oregon pine
Modulus of elasticity [kN/m^2]	1.232E+07	9.327E+06
Density [kg/m^3]	492	344
Poisson Modulus	0.3	0.3

5.1.2 Restraint and Loading Conditions

To simulate the base restraint conditions, for all the models, the nodes from the bottom beam are restrained in the vertical and horizontal directions. In order to apply the prescribed horizontal displacement for the analysis, the top beam is restrained in the horizontal direction.

The loading conditions applied to the model include a vertical load and a prescribed horizontal displacement. The horizontal displacement was of 0.1 m, applied in increments for the nonlinear analysis. While the vertical load included the dead load of the beams, the joists and timber boarding of each floor. The total vertical load was calculated based on the structural survey carried out by Jiménez (2015). It was assumed that the internal walls are spaced at 4 m and that the slabs act in one direction. The loading guide for typical house loadings from evolution was used as a reference to obtain the floor loads shown in Table 5.2. The loading conditions varied for the two main representative cases of study that will be described in the following sections. For Case 1 the calculated tributary area was 22.84 m^2, while for Case 2 was 223.34 m^2. Due to this the loads to be consider per floor for each case resulted to be different. Finally, an occupancy load of 0.6 kN/m^2 was considered for both cases. Table 5.3 show the loads to be consider per floor for each case.

Table 5.1.2 Floor loads considerations. (evolution, n.d.)

Joists (150mm x 50mm @ 450mm c/c)	0.12 kN/m^2
Timber boarding	0.07 kN/m^2

Table 5.1.2 Distributed load to be consider per floor for each case of study.

Case	Case 1	Case 2
Joists	2.74 kN	2.80 kN
Timber boarding	3.20 kN	3.27 kN
Oak weight	0.89 kN	1.05 kN
Occupancy	13.7 kN	14 kN
Total distributed load	**1.67 kN/m**	**1.46 kN/m**

5.2 Application to Representative Cases of the Historical Center of Valparaiso

The finite element software SAP2000 was used for the numerical models of timber frames from Valparaiso, since it provides a simpler application for modelling the nonlinear behavior of the carpentry joints. Before the model is analyzed it is calibrated to obtain the nonlinear behavior caused by the carpentry joints explained and described in the previous chapter. Since there are no previous experimental or analytical studies of the timber frames from Valparaiso, other experimental works were used as a reference for the calibration process. These are the Pombalino timber frame (Poletti, 2013) and the Quincha timber frame (Quinn, 2017), both described in Chapter 2 and explained for the calibration process in Chapter 3. Both frames have similarities with the timber frames from Valparaiso, specifically with the carpentry joint typologies. The Pombalino timber frame is used as a reference to relate the behavior of the connection by contact found in the timber frame from Valparaiso. However, the Quincha timber frame is used as a reference to relate the behavior of the mortise and tenon connection, and the notched connection.

To model the connection by contact, the approach used by Ciocci (2015) in the analytical model of Pombalino was considered. To adjust the behavior of the joint to the case of Valparaiso, the component method was applied to obtain the appropriate contact stiffness by the following equation (5.1),

$$k_c = \frac{E_\alpha \sqrt{b\ d}}{c} \tag{5.1}$$

where, b and d are the dimensions of the contact area, E_α is the modulus of elasticity according to the direction to the grain, and c is a coefficient depending on the ratio between b and d, and the coefficient of Poisson.

The modulus of elasticity was evaluated according to equation (5.2),

$$E_\alpha = \frac{E_0 * E_{90}}{E_0(Sin\ \alpha)^2 + E_{90}(Cos\ \alpha)^2} \quad (5.2)$$

where E_0 and E_{90} are the parallel and perpendicular moduli of elasticity and α is the grain direction.

Assuming that Oak has a strength class and characteristic values classified as D30 according to EN 338, E_0 is 10 kN/mm^2 and E_{90} is 0.64 kN/mm^2. Finally, considering that $\alpha = 71°$, the modulus of elasticity resulted in 7.10E+05 kN/m^2.

To obtain the contact stiffness a contact area of 0.0237 m^2 was considered and the stiffness resulted in 9.117E+04 kN/m. Finally, to obtain the yielding force and the yielding displacement equations 5.3 and 5.4 were applied considering $f_{90} = 8$ N/mm2. F_y resulted in 189.72 kN and d_y resulted in 0.00208 m. By assuming an ultimate displacement of 0.05 m the force-deformation relation, shown in Figure 5.1, was obtained.

$$F_y = A_c * f_{90} \quad (5.3)$$

$$d_y = \frac{F_y}{k} \quad (5.4)$$

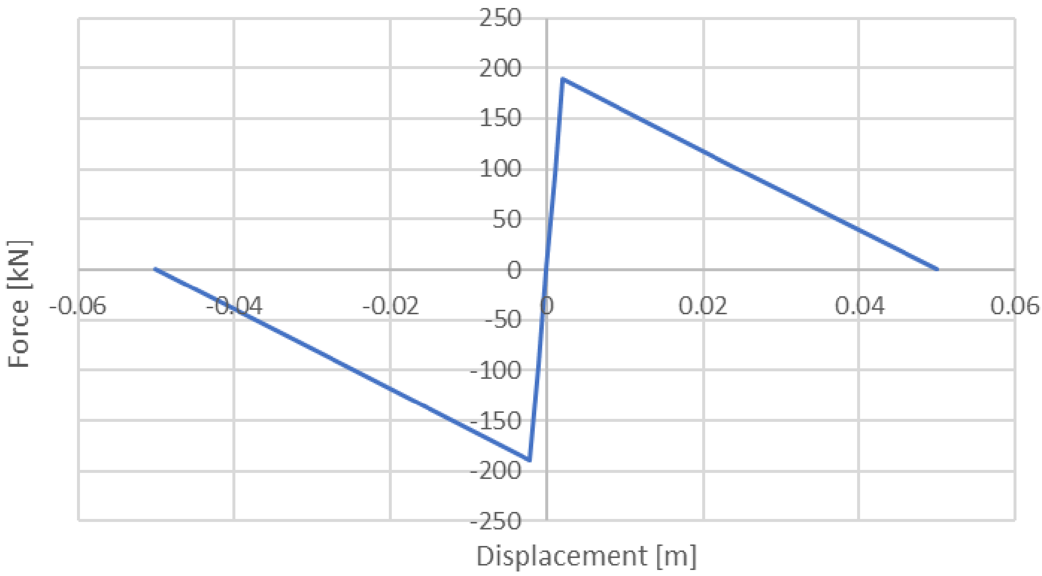


Figure 5.2 Force-deformation for the connection by contact.

To model the mortise and tenon connection, the approach proposed by Quinn (2017) for the Quincha timber frame model was considered. To adjust the behavior of the joint to the case of Valparaiso the relation proposed by Quinn between the tenon geometry and the rotational stiffness, shown in Figure 5.2, was used. Considering a tenon length of 50 mm the rotational stiffness resulted in 8 kNm/rad and the moment-rotation relation, shown in Figure 5.3, was obtained. For the translational stiffness the stress-strain relation described by Quinn was assumed for the behavior of the mortise and tenon connections, as shown in Figure 5.4.

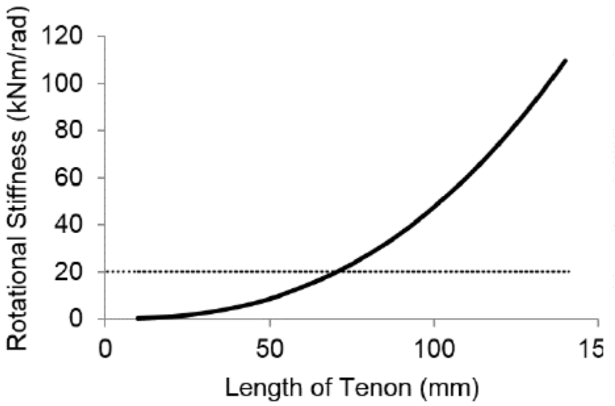


Figure 5.2. Variation in stiffness with length of tenon. (Quinn, 2017)

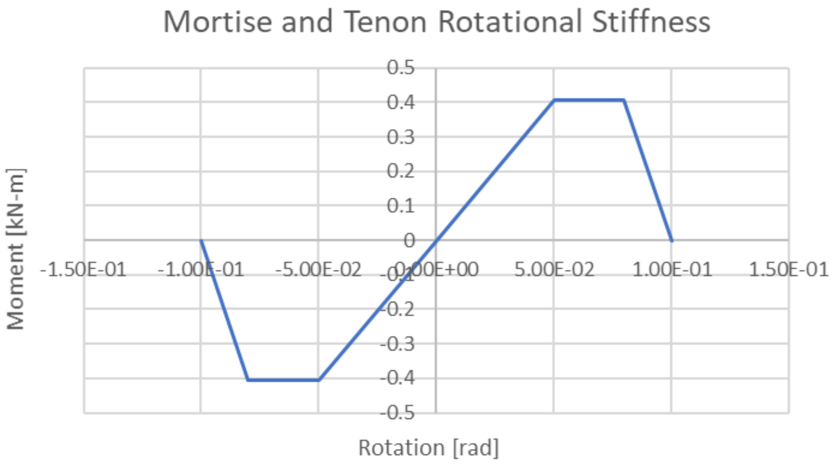


Figure 5.2 Moment-rotation diagram for the mortise and tenon connection.

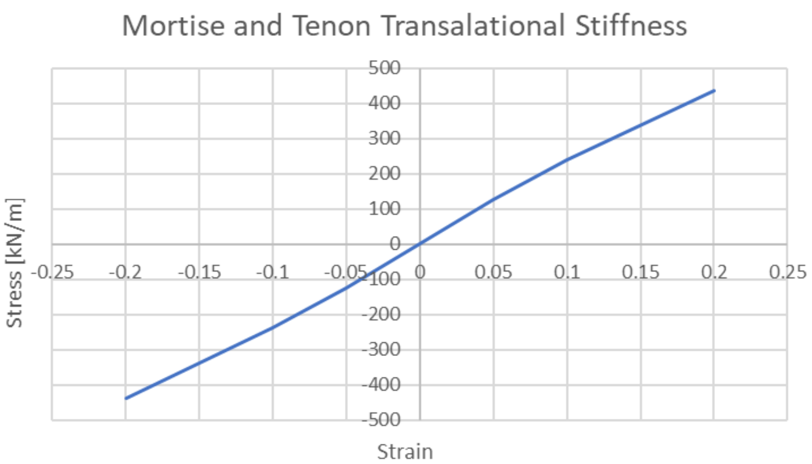


Figure 5.2 Stress-strain relationship for the mortise and tenon connection.

To model the notched connection, the approach proposed by Quinn (2017) for the Quincha timber frame model was considered. To adjust the behavior of the joint to the case of Valparaiso Eurocode5 was used to calculate the capacity of the connection. Considering laterally loaded nails in single shear with failure mode (f) (Figure 5.5), the following equation (5.5) was used to calculate the capacity:

$$1.15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk}f_{h,1,k}d}+\frac{F_{ax,Rk}}{4} \tag{5.5}$$

where $\beta=\frac{\mathrm{f_{h,2,k}}}{\mathrm{f_{h,1,k}}}$, $M_{y,Rk}=0.3f_u d^{2.6}$ for rounded nails, and $f_{h,1,k}=0.082\rho_k d^{-0.3}$ for no predrilled holes.

Considering two nails with 5 mm diameter the capacity of the connection resulted in 2.28 kN and a force-deformation relation, shown in Figure 5.6, was obtained.

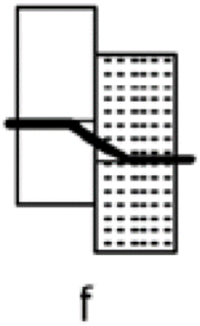


Figure 5.2 Failure mode (f) for laterally loaded nails in single shear.

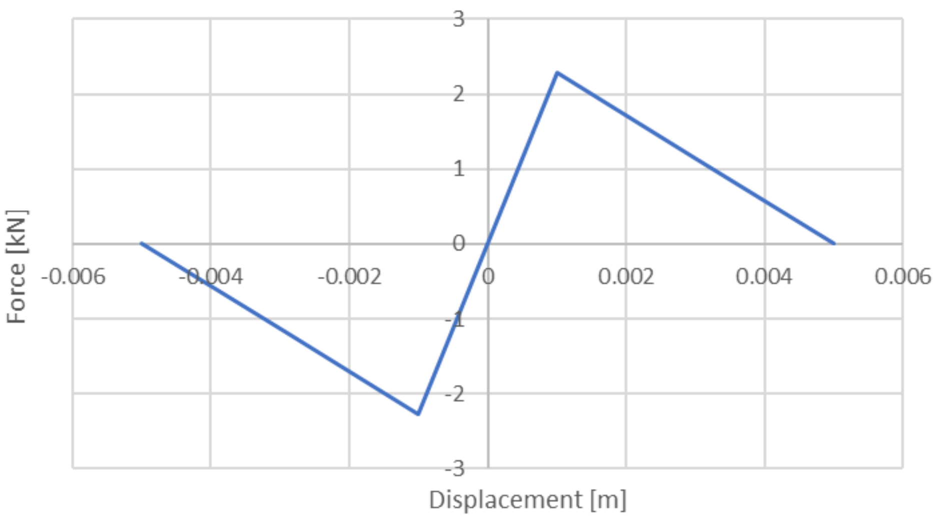


Figure 5.2 Force-deformation diagram for the notched connection.

The next sections will describe the numerical analysis performed for the representative cases of timber frames in Valparaiso by applying these assumptions and considerations. Two main representative cases of the historical center of Valparaiso are analyzed, but other basic frame analyses are performed first. These included the analysis of the timber frame elementary cell, a basic frame with a window opening and a basic frame with a door opening.

5.2.1 Elementary Cell

The elementary cell or modulus that configures a typical load-bearing walls is composed by two beams, three posts and one diagonal as described in Section 4.2.1. The elementary cell was modeled by applying the geometry, material properties, restrains and loading conditions described in the previous sections. The numerical model was done for the diagonal working in tension, as shown in Figure 5.7(a). The carpentry joints behavior described previously, were applied by using hinges at the location of the joints, as shown in Figure 5.7(b). An axial load release was applied at the upper end of the diagonal element to simulate the ability of the notched joint to move axially. Also, a moment release was applied at the notched connection between the diagonal and the internal post. The obtained pushover curve of the elementary cell is shown in Figure 5.8. The curve shows that nonlinear behavior starts to take place when the lateral load reaches 0.7 kN and that the maximum load at 0.1 m displacement is 3 kN.

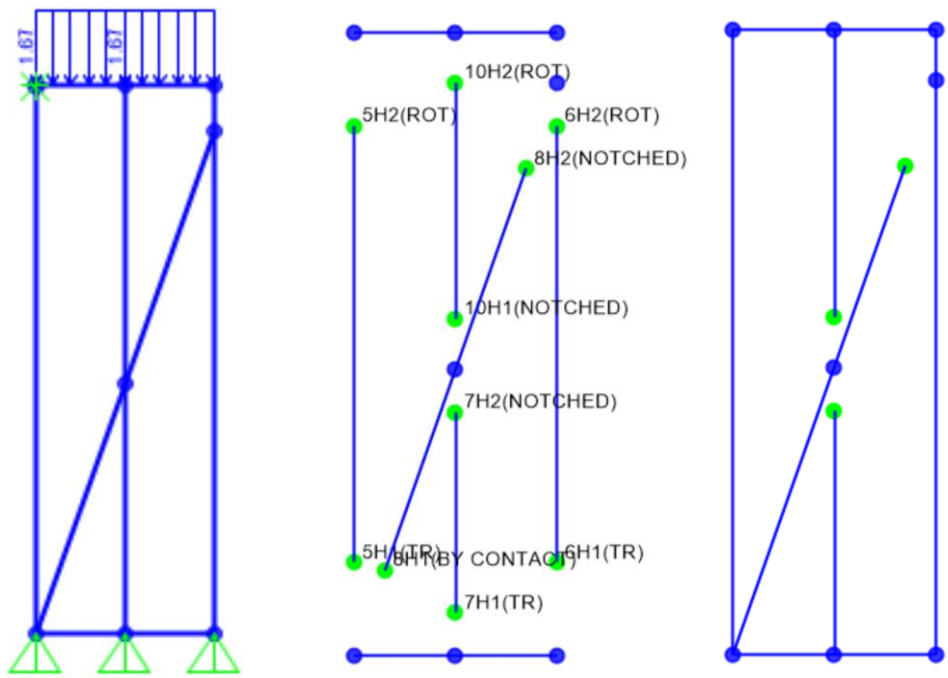


Figure 5.2.1 Elementary cell: (a) Numerical model. (b-c) Hinges and Releases.

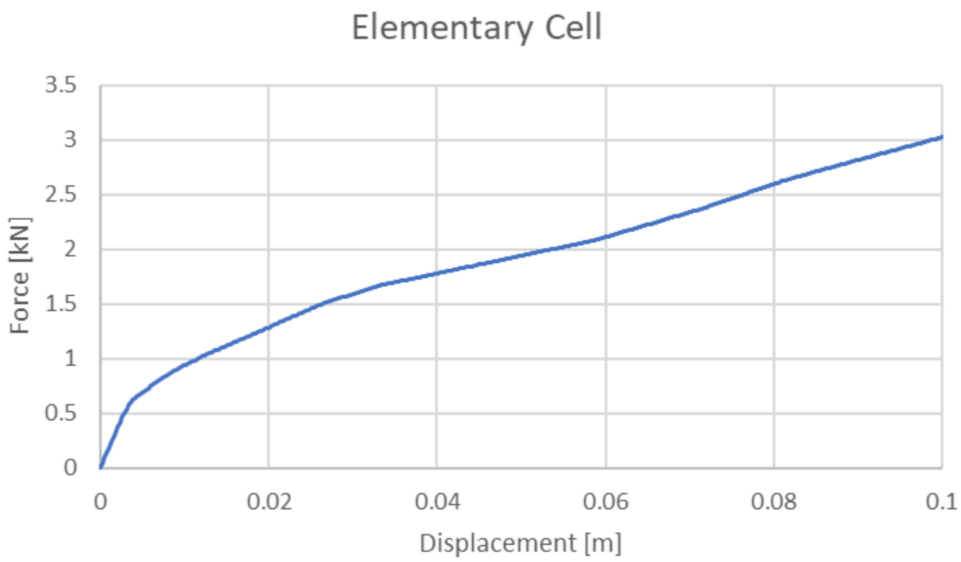


Figure 5.2.1 Numerical pushover curve of Valparaiso timber frame elementary cell.

5.2.2 Basic Frame with Door Opening

The basic frame with a door opening is composed by two elementary cells, one at the left and one at the right of the door opening as described in Section 4.2.2. The frame was modeled by applying the geometry, material properties, restrains and loading conditions described in the previous sections. The carpentry joints behavior described previously, were applied by using hinges at the location of the joints, as shown in Figure 5.10(a). The same release conditions applied for the elementary cell were considered, as shown in Figure 5.10(b). The obtained pushover curve of the basic frame with a door opening is shown in Figure 5.11. The curve shows that nonlinear behavior starts to take place when the lateral load reaches 2 kN and that the maximum load at 0.1 m displacement is 14 kN.

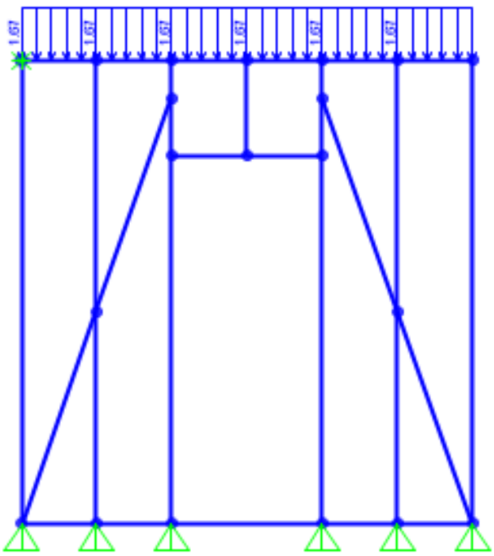


Figure 5.2.2 Basic frame with door opening numerical model.

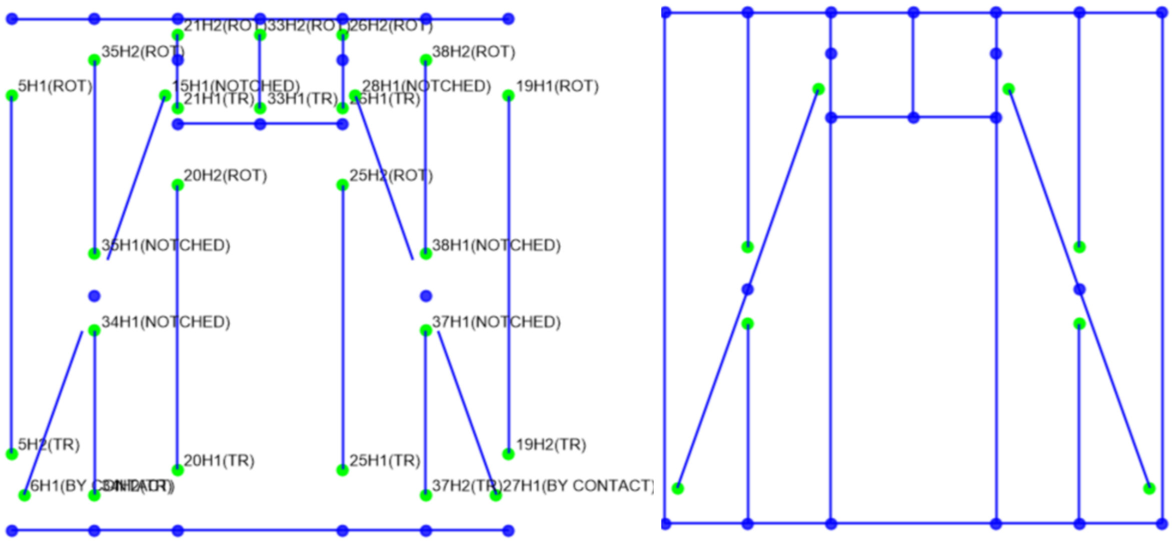


Figure 5.2.2 Basic frame with door opening: (a) Hinges. (b) Releases.

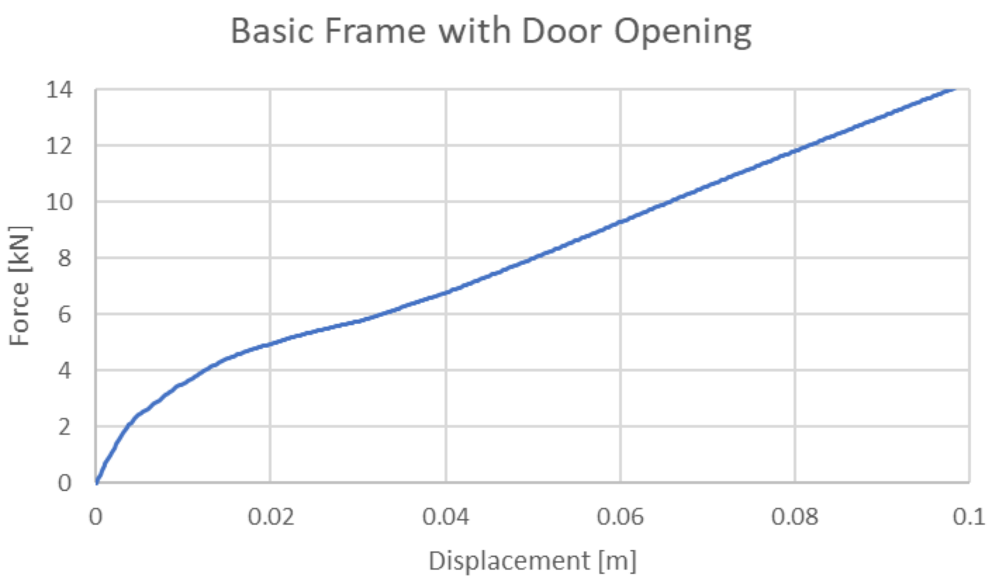


Figure 5.2.2 Numerical pushover curve of Valparaiso timber frame with door opening.

5.2.3 Basic Frame with Window Opening

The basic frame with a window opening is composed by two elementary cells, one at the left and one at the right of the window opening as described in Section 4.2.2. The frame was modeled by applying the same conditions and assumptions already mentioned. The carpentry joints behavior described previously, were applied by using hinges at the location of the joints, as shown in Figure 5.13(a). The same release conditions discussed previously were applied, as shown in Figure 5.13(b). The obtained pushover curve of the basic frame with a window opening is shown in Figure 5.14. The curve shows that nonlinear behavior starts to take place when the lateral load reaches 2 kN and that the maximum load at 0.1 m displacement is 14 kN.

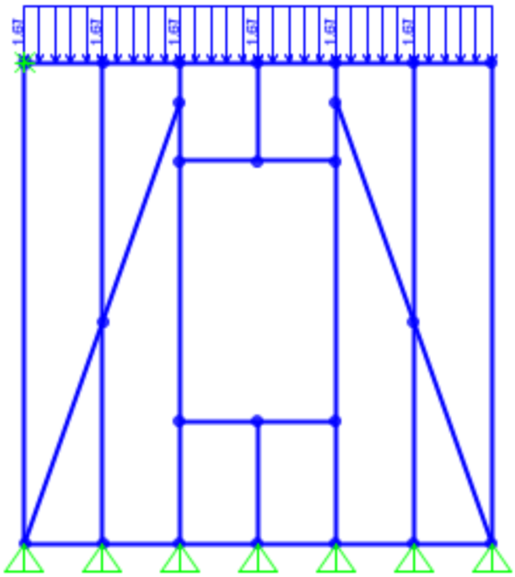


Figure 5.2.3 Basic frame with window opening numerical model.

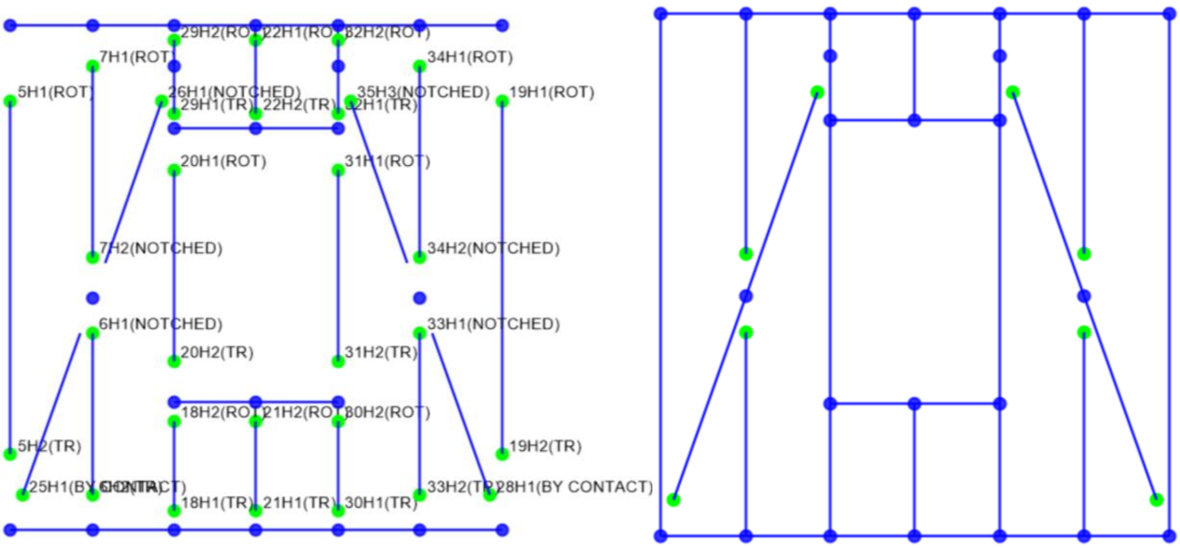


Figure 5.2.3 Basic frame with window opening: (a) Hinges. (b) Releases

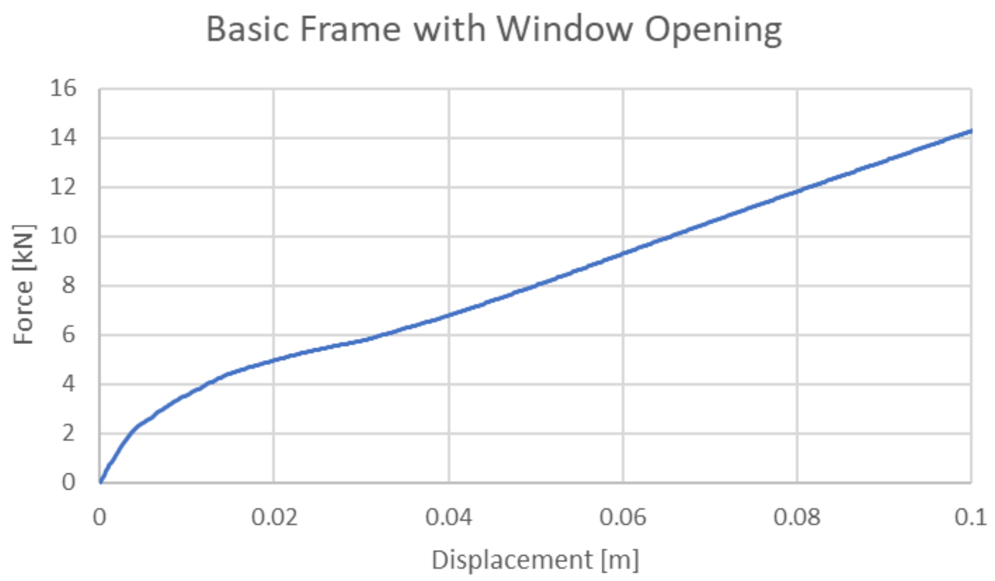


Figure 5.2.3 Numerical pushover curve of Valparaiso timber frame with window opening.

5.2.4 Representative Cases

Two representative cases of typical houses from Valparaiso were selected for analysis. Typical characteristics of these houses are how they are built in a sloped terrain and how the structure configuration is adjusted to the surroundings. The plan configuration is characterized by being regular but adjusted to the surrounding limits. The main façade usually has several window openings and details from the Victorian style. The structural configuration of these houses can result to be very similar. Some variations are the number of storeys, the location of the bracing elements, and the quantity of window and door openings. It results interesting to study how the response of the structural system is dependent of these variations.

The first case of study (Case 1) is **Montealegre #138**:

The house from Montealegre #138, shown in Figure 5.15, is characterized by two storeys configured with timber frames. For this study, the main façade timber frame wall will be analyzed. The composition of the wall includes two storeys with three window openings at the top floor, and two window and one door opening at the lower floor. The frame is composed of beams, posts and diagonal bracings, as shown in Figure 5.16(b). To develop the numerical model, the same characteristics and assumptions used for the simple models (elementary cell, door opening and window opening) were considered. Figure 5.17 shows the setup of the numerical model including the loading and boundary conditions. Figures 5.18 and 5.19 show the location of the application of the hinges for modeling the nonlinear behavior of the connections, and the releases considered. The obtained pushover curve of the shear wall is shown in Figure 5.20. The curve shows that nonlinear behavior starts to take place when the lateral load reaches 7 kN and that the maximum load at 0.1 m displacement is 20 kN.

Figure 5.2.4 Case 1: Montealegre #138. (Jiménez, 2015)

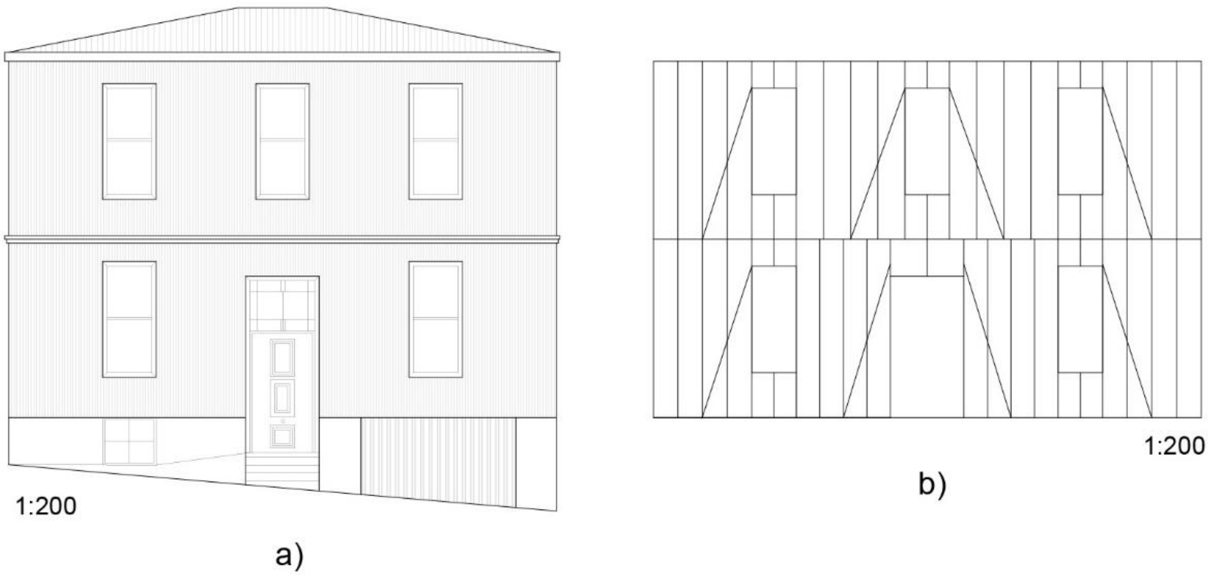


Figure 5.2.4 Case 1 Montealegre 138 (a) main façade. (b) modelling idealization. (Jiménez, 2015)

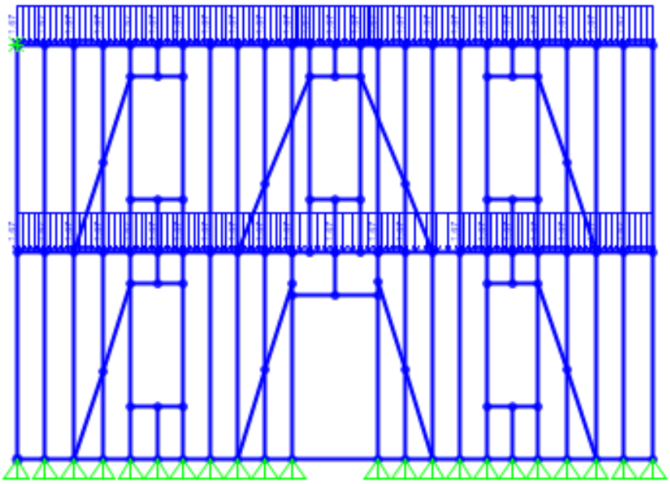


Figure 5.2.4 Case 1 Montealegre 138: numerical model.

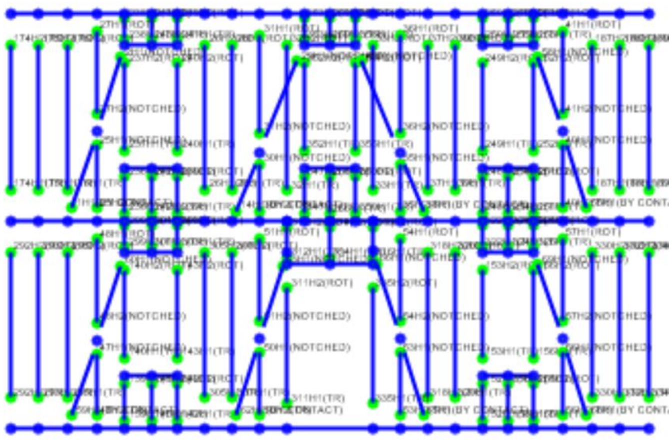


Figure 5.2.4 Case 1 Montealegre 138: hinges.

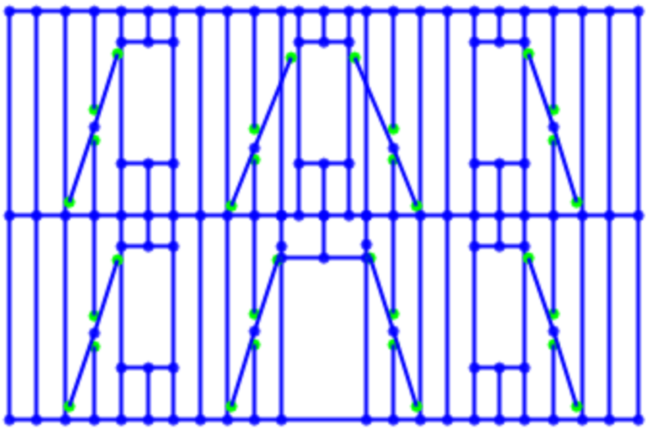


Figure 5.2.4 Case 1 Montealegre 138: releases.

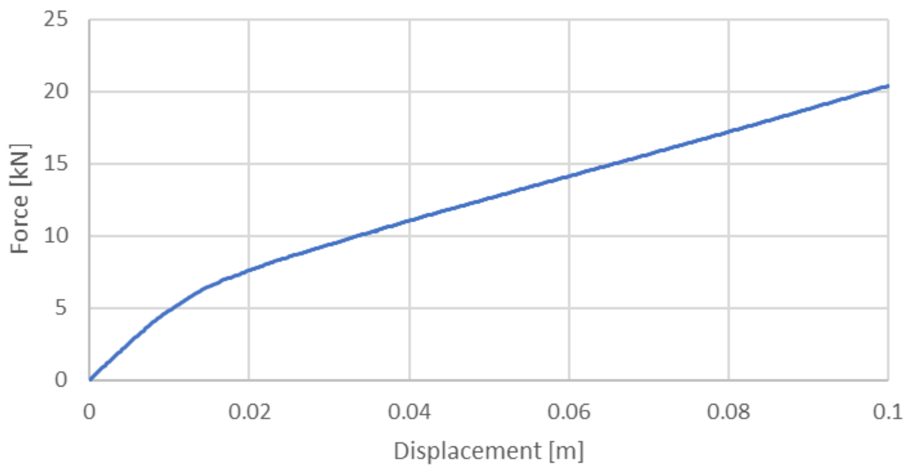


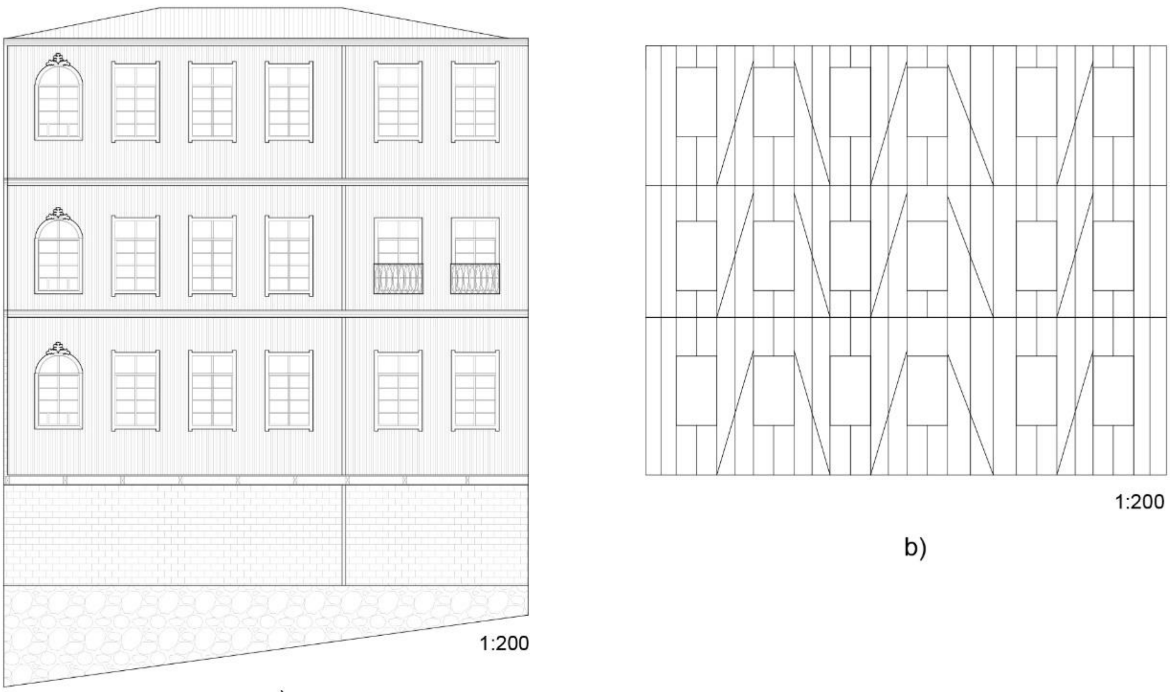
Figure 5.2.4 Case 1 Montealegre 138: numerical pushover curve.

The second case of study (Case 2) is **Almirante Montt #491:**

The house from Almirante Montt #491, shown in Figure 5.21 and 5.22, is characterized by three storeys configured with timber frames. For this study, the main façade timber frame wall will be analyzed. The composition of the wall includes three storeys with six window openings at each floor. The frame is composed of beams, posts and diagonal bracings, as shown in Figure 5.23(b). To develop the numerical model, the same characteristics and assumptions used for the simple models (elementary cell, door opening and window opening) were considered. Figure 5.24 shows the setup of the numerical model including the loading and boundary conditions. Figures 5.25 and 5.26 show the location of the application of the hinges for modeling the nonlinear behavior of the connections, and the releases considered. The obtained pushover curve of the shear wall is shown in Figure 5.27. The curve shows that nonlinear behavior starts to take place when the lateral load reaches 8 kN and that the maximum load at 0.1 m displacement is 22 kN.

Figure 5.2.4 Case 2: Almirante Montt #491. (Jiménez, 2015)

Figure 5.2.4 Case 2: Almirante Montt #491. (Jiménez, 2015)



a) b)

Figure 5.2.4 Case 2: Almirante Montt 49 (a) main façade. (b) modelling idealization. (Jiménez, 2015)

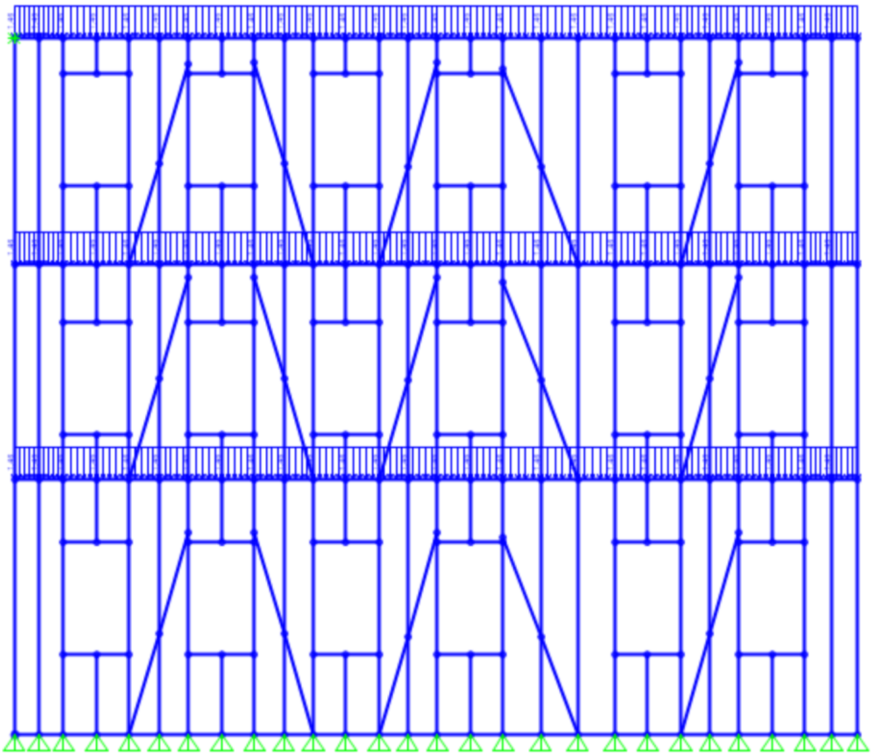


Figure 5.2.4 Case 2 Almirante Montt 49: numerical model.

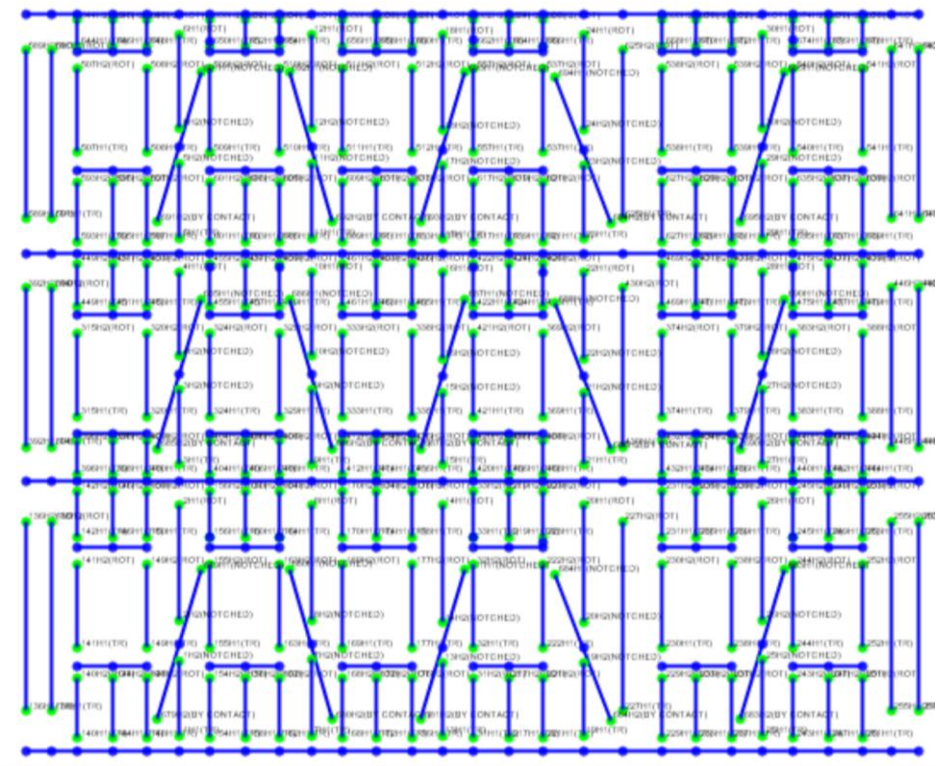


Figure 5.2.4 Case 2 Almirante Montt 49: hinges.

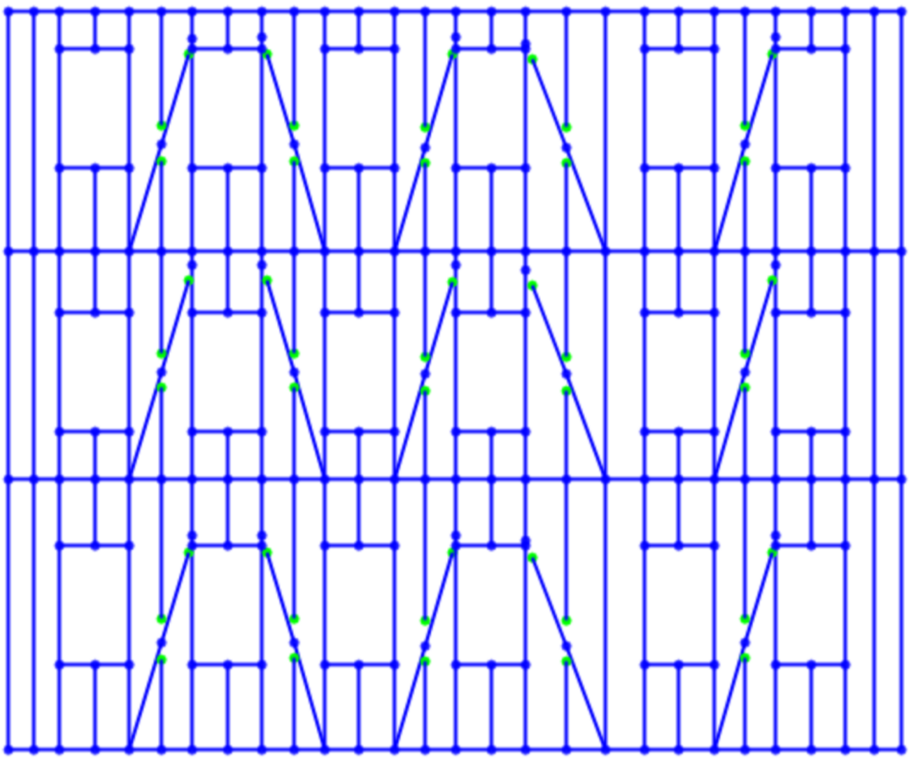


Figure 5.2.4 Case 2 Almirante Montt 49: releases.

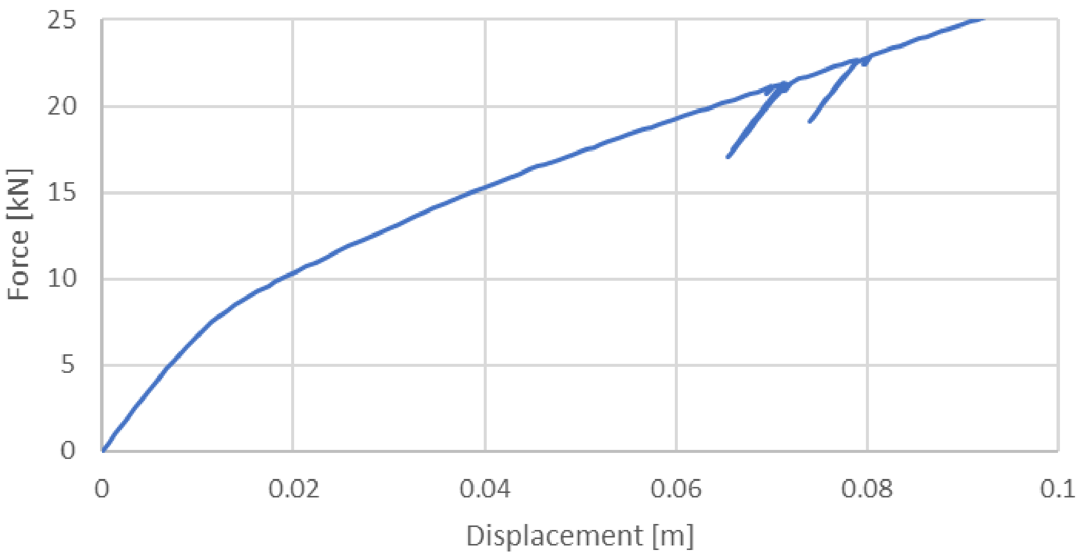


Figure 5.2.4 Case 2 Almirante Montt 49: numerical pushover curve.

CHAPTER 6 CONCLUSIONS

6.1 Summary

In order to represent the behavior of a traditional timber frame wall, two-dimensional models were developed following the lumped plasticity modelling approach and taking into account the nonlinearities produced by the carpentry connections. The analysis performed was a nonlinear static (pushover) analysis. Two experimental campaigns were studied in order to perform a valid modelling calibration that can be applied to a timber frame wall without previous studies. Two modeling softwares, SAP2000 and DIANA FEA, were used in order to validate the calibration. Finally, two cases of timber frames from Valparaiso, Chile, were studied and analyzed. The models were able to capture the stiffness of the timber frame walls, the deformation, the nonlinear behavior and the expected mechanisms that where based in the calibrations of Pombalino and Quincha frames. However, a more detailed study is needed to obtain more accurate results.

6.2 Outcomes of the Study

The analysis performed on timber frame walls demonstrated the importance of the connections and their influence in the global behavior of the frame. It was understood, that the connections are the weakest location of the frame, where failure can occur. Based on the research performed for this study, it was concluded that the behavior of the connections will depend of the type of joint, the constructive technique, the geometry, the wood properties, and their location in the frame.

The connections behavior was modeled by a calibration process based in previous experimental campaigns. The outcome of the numerical calibration resulted with a good approximation compared to the experimental results. Since the model calibration was performed by two softwares, SAP2000 and DIANA FEA, part of the outcome of this study is to identify the advantages and disadvantages for their application in each software. SAP2000 resulted to provide a more user-friendly approach for the modelling of the nonlinear behavior of the carpentry connections, by using concentrated nonlinear hinges. Meanwhile, in DIANA FEA the nonlinearities were represented by spring elements that needed to consider assumptions like: the size of the spring, their working direction, and the way the elements that were connected are tied. The application in this case can result more time consuming and less accurate. It was concluded that the selection of the software to be used for modeling is very important and it will depend of the expected outcome. An advantage of SAP2000 is the simplicity and time efficiency in the case of applying the connections behavior to the model. However, an advantage of DIANA FEA is that it provides the required tools to model the interaction between different materials. This will be useful to further develop a numerical model that not only considers the nonlinearity of the connections, but also the interaction between timber and masonry.

The calibration of the models resulted to be a very important step in this study, since the numerical models of typical typologies of timber frames form Valparaiso were based on this calibration. The outcome of this study was to contribute to the development of a numerical model of a real case study for which there is no previous experimental research. The purpose for the development of these models was to create a methodology that can be applied to similar cases where there is no previous information about the structural behavior of the frame.

The work carried out seek to search for a correct approximation to model timber frame structures in cases where there are no experimental studies that allow to obtain more accurate results from the numerical models. Is important to mention that results obtained for the cases of Valparaiso are qualitative results, since only the geometry of the real case study was considered, and the mechanical behavior of the joints was assumed based in experimental studies of similar cases. A more critical analysis of the mechanisms of this specific typology and a more detailed calibration of the connections is needed to obtain more accurate results. It was noted that the capacity obtained from the models of the walls of Valparaiso resulted low when compared with the behavior of other timber frames like for example Pombalino. The behavior of the wall can be improved if other detailed studies are carried out for the calibration of the model.

6.3 Suggestions for Future Research

This topic results very interesting and with a high possibility of extension for research. The numerical calibrations and analyses performed in this study are a contribution to the analysis of traditional timber frame walls. One of the further developments could be to perform a detailed study of the timber frames from Valparaiso and from the joints that characterize the frame by performing experimental works. This could lead to less qualitative and more accurate results for the mechanisms of the walls. Another interesting option could be to develop more detailed analytical studies of the carpentry joints, in order to obtain a more accurate calibration of the model.

The timber frame typologies studied in this thesis are characterized by having infill and interaction with masonry walls. However, for this study only the timber frame was considered. It could be interesting and useful to develop a numerical model where the interaction between the two materials (timber and masonry) is included. The infill provides additional stiffness to the frame and there are many variations of the materials used, depending of the typology and region. An interesting further research could be to study how the variations of materials, based on their mechanical characteristics, affect the stiffness and global behavior of a wall.

Another possibility of development is to study the seismic vulnerability of the timber frame. For this purpose, the following procedure is suggested:

(a) Study and analyze different structural typologies of timber frames.
(b) Apply the spectrum capacity method to determine the seismic response of the structure subjected to different seismic scenarios.
(c) Calculate the fragility curves to obtain a probabilistic study of the levels of damage of the structure, considering different seismic demands.

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